REPORT

ON THE

RENEWAL

OF

NIAGARA SUSPENSION BRIDGE.

BY

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To the Presidents and Gentlemen of the Boards of Directors of the Niagara Falls Supension and the Niagara Falls International Bridge Companies:

GENTLEMEN:

Having completed the work of reinforcing the Anchorage and renewing the Suspended Superstructure of your bridge, I respectfully submit the following report.

LEFFERT L. BUCK.

December 13th, 1880.

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REPORT

ON THE REINFORCEMENT OF THE ANCHORAGE AND RENEWAL OF THE SUSPENDED SUPERSTRUCTURE OF THE NIAGARA RAILROAD SUSPENSION BRIDGE.

From the inception of the project of spanning the chasm of the Niagara River below the Falls, with a suspension bridge for railroad purposes, to the year 1855, when the bridge was completed and opened to traffic, it was considered by all as a bold undertaking and by some engineers, even, as an impracticable one. But the bridge has been in constant use for twenty-five years and under constantly increasing traffic, thus demonstrating its adaptability to a locality requiring such long spans.

In spite of its success, however, it has been an object of constant solicitude to the traveling public. The frightful chasm that it spans would naturally excite the fears of most people, but this has been greatly enhanced by doubts as to the condition of the cables and their anchorage.

The object of this Report is to show what the real condition of the cables and anchorage was, and also to indicate what has been done for their improvement, as well as that of other parts of the bridge.

In order to explain this clearly, it is necessary to insert here the following general description of the structure as it was.

DESCRIPTION.

The bridge consisted of the following members:

1st. Two pairs of iron wire cables resting on masonry towers at each end of the bridge. The ends of the cables are secured by means of chains to suitable cast iron anchor plates bedded in the rock forming each bank of the river. Two of these cables have, at mean temperature, a versedsine of 54 feet, and are designated upper cables. The other pair having a versed sine of 64 feet, are called the lower cables.

2d. The Suspended Superstructure.

This consisted of two floors, placed at a vertical distance apart of 17 feet and connected by posts and rods in such a manner as to form a trussed tube.

At each five feet in the length of the trusses, two wire rope suspenders connect the upper floor with the upper cables. In the same manner the lower floor is suspended to the lower cables.

CONSTRUCTION OF THE CABLES.

Each cable is composed of seven strands or bundles of wire. Each strand is made up of 520 scant No. 9 wires laid parallel, and at each end formed into a loop which fits into a groove in a U shaped cast iron shoe. The seven strands are bound into one bundle, of 3,640 wires, which is served closely with wire over the whole length, with the exception of about 13 feet, at each end, and of about 10 feet of the portions resting on the towers, thus forming a cylindrical cable $10\frac{1}{8}$ inches in diameter.

The tops of the towers are each covered with a cast iron plate, 8 feet square, bedded in mortar. The upper surface of this plate is planed to a true surface. On it rest a number of turned iron rollers 5 inches in diameter. On these rollers rest the saddles, consisting of heavy castings, whose undersides are planed. The top of each saddle has a groove of semi-circular section in which the wires of the cable lie, each cable having a separate saddle. The planes of the curves of the cables, between the towers, are inclined in such a manner as to bring those of each pair nearer together at the middle of the span, to give lateral stability to the bridge. From the towers to the anchorage the cables diverge from the center line of the bridge sufficiently to make the plane, containing the portion each side of the tower, vertical. The wire forming the cables was boiled in linseed oil before it was laid, and as the cables were made the interstices at the shoes and towers were flushed with boiled linseed oil and Spanish brown paint. Then the whole length of the cable was flushed with the same as the serving progressed.

ANCHORAGE.

Each end of each cable has a separate anchorage the description of which will be assisted by referring to Plate 1.

A rectangular pit or shaft, 3 ft. x 7 ft. in plan, was sunk vertically into the rock, to a depth of 25 feet, with the bottom enlarged to form a chamber 7 ft. square. An anchor plate, 6 ft. 6 in. square and having seven rectangular openings through it to receive the lower links of the anchor chain, is set in the chamber, the links put in position and secured by a $3\frac{1}{2}$ inch diameter pin passed through their heads and underneath the plate. From the plate the chain passes vertically upward to the surface of the rock. From this point the joints of the chain are at points of a vertical curve of 25 ft. radius. The joint at the upper end of the curve forming the point of tangency with the line of the cable. Beyond this joint is another length of chain composed of nine links, each bar of which is 10 feet long and 7 in. x $1\frac{5}{8}$ in. section. Four of these links alternate with the shoes of three of the strands of the cable and are secured to them by a $3\frac{1}{2}$ in. diameter pin passing through links and shoes. The remaining five links are in like manner connected with the remaining four shoes of the cable strands, as shown in elevations (Figs. 1 and 2).

The anchor plates are secured in the chambers by means of neatly fitted stone blocks set in cement mortar, the whole pit being solidly filled with cement masonry and the interstices around the bars grouted. Above the rock and up to the end of the chain the whole is enclosed in a solid wall of masonry, heavy blocks of which form supports of the joints of the curved portion of the chain. Formerly the strands were also covered with masonry and the whole grouted, the intention being to preserve them from corrosion.

CONSTRUCTION OF THE OLD SUSPENDED SUPERSTRUCTURE.

As first built the superstructure was as follows (see right hand half of transverse view and side elevation of old truss, plate IV.)

1st. A series of transverse bents, one at each five feet in the length of the bridge, or 161 in all. Each bent consisted of the following parts:

a. Lower floor beam. This was made of two pieces of 4 in. x 16 in. pine timber, 24 feet long, set on edge, parallel and with a 6 in. space between them.

b. Two posts. Each post was composed of two pieces of 4 in. x 6 in. pine placed parallel and separated 4½ inches by packing blocks. The lower ends of the posts were secured between the two parts of, and flush with, the lower edge of the lower floor beam. The clear distance between the posts of a bent was 19 feet.

c. Upper floor beam. The upper beam was the same as the lower, except that it was 25 feet long. The top of the post was flush with the top of the beam and clamped between its two parts.

d. Knee braces. Each bent was stiffened transversely by a 4 in. x 6 in. pine knee brace from the under side of the upper beam to the inner face of the post.

2d. Longitudinal members.

a. Upper and lower chords.

The upper chords each consisted of two thicknesses of 2 in. x 12 in. white oak plank in lengths of 5 feet, the joints of one thickness being at the middle of the lengths of the other. They were bolted together with eight ½ in. bolts to each length. These chords laid on tops of posts and beams.

The lower chords were each made of three thicknesses: one of pine 2½ in. x 12 in., one of pine 4 in. by 12 in., and one of oak 2 in. x 12 in., all in 5 feet lengths and bolted together with eight ½ in. bolts to each panel. The lower chords were laid underneath the ends of the posts and the undersides of the lower floor beams.

b. Track Stringers.

There were two deep longitudinal track stringers built partly above and partly below the upper floor beams. The portion above the floor beams was made of 4 in. x 15 in. pine planks piled closely, broadsides horizontal and to a height above the beams of 2 ft. 6in. The rails for the track were

laid on the tops of these stringers. The portion of each stringer below the beams was made of two pieces of pine timber, one piece being 12 in. x 12 in. and the other 12 in. x 10 in. They were made continuous by scarfed and keyed splices. The spaces between the upper and lower portions of the stringers and from one beam to another were filled by bridging pieces 4 in. thick. There were 5 long bolts (1 inch diameter) to each panel length of stringer extending through from top to bottom.

c. Hand rails. At each side of the upper floor was a heavy. trussed hand rail, extending the whole length of the bridge. At each beam a $1\frac{1}{8}$ in. diameter bolt passed from the top of the hand rail down between the parts of the beam and through an 8 in. x 9 in longitudinal pine string piece. This string piece was made continuous by splicing.

d. Upper floor. The planking of the upper floor was laid lengthwise and fastened to the beams by means of screws. The portion between the track stringers was of 4 in. plank, the remainder was 2 in. thick.

e. Lower floor planking. The first course of plank was of pine 2 in. thick. The top course of oak 2 in. thick. Both courses were laid lengthwise and secured to the beams with screws.

f. Cornices. To the ends of the upper and lower floor beams were secured strong cornices.

3d. Truss rods. The truss rods were at first made of 1 in. diameter round iron with a nut on each end. They extended each way from the bottom of each post diagonally to the top of the fourth post distant, passing through the chords and secured by the nuts being screwed down to the face of a cast-iron angle washer.

4th. Suspenders. The lower cable suspenders were attached by means of stirrup to the ends of the lower floor beams.

Those of the upper cables were attached to the upper floor beam on each side, about midway between the track and the trusses.

5th. Stays.

a. From the top of each tower, where one end of each was

secured to the saddle, sixteen wire rope stays extended to various points of the upper and lower floors. They were of wire rope 4½ in. circumference. The longest reached to a point 250 feet from the end of the truss.

b. Attached to each side of the lower floor, at intervals of 25 feet for the whole length of the bridge, excepting 75 feet at each end, were wind stays whose other ends were fastened to the rocks on each side of the river. There are fifty-six of them in all.

CHANGES IN THE OLD WORK.

After the bridge had been in use for some time, it was found that the lower floor did not assist the chords until the latter had been overstrained, after which the constant working of the joints at the intersection of posts with beams and chords, had produced rapid decay of these parts. As they became weakened and the stress came upon the floor planking, the screws holding them to the beams were loosened, allowing moisture to get in and causing the upper edges of the beams to rot.

To remedy this difficulty, auxiliary chords, composed of heavy timber, were bolted to the undersides of the lower floor beams, parallel with, and just inside of the truss rod chords.

To obviate the necessity of removing the old beams, their projecting ends were cut off, the posts were cut off level with the tops of the beams, the portion between the beams removed, and pieces of 6 in. x 13 in. pine timber inserted from each side of the bridge, their inner ends meeting at the center line and their outer ends projecting to form an attachment for the lower suspenders and to which the lower cornices were attached. The lower ends of the posts then rested upon the new beams and oak shoulder-blocks were spiked to the sides of beam and post. The bolts holding the auxiliary chords passed up through these new beams and remaining portions of the old beams (between which the new ones were placed) were secured by bolts passing through all three.

Such was snbstantially the arrangement of the parts of the bridge up to the year 1877. But the upper floor had been renewed in 1873, though no alteration was made in the arrangement.

When in the condition above described, the whole suspended weight, between the towers, including cables, stays and suspenders, was estimated at 1,180 tons.

INSPECTION.

In the latter part of February, 1877, Mr. Thomas C. Clark, of the Phœnixville Bridge Co., with a view to examining the condition of the portions of the cable strands embedded in the masonry, caused a small excavation to be made near one of the shoes. On reaching the first strand, two or three of the wires were found to be corroded quite through and others were partially corroded.

Shortly afterward Col. W. H. Paine, of the East River bridge, visited the bridge and gave orders for the removal of all the masonry covering the strands of each cable. He also made tests of the elongation of the strand portion of one of the cables, by means of a vernier scale. He found in this way that the elongation under a given moving load, on the bridge, was no greater than the modulus of the wires would allow, supposing the total section to be the same as when the cables were new. He also cut out some pieces of wire and tested them for tensile strength, ductility, &c.

Their ultimate strength was fully equal to that of the new wire per unit of section, and their reduction of ruptured section was satisfactory, but as the wires tested were etched in places, of course the stretch would be principally confined to the etched portion, hence rendering any measurement of the stretch a matter of extreme difficulty.

On the 16th of March, 1877, the writer joined Col. Paine at Suspension Bridge to assist in examining the condition of the bridge and in repairing the defective wires.

After the strands were thoroughly cleaned and the wire bands removed, they were opened, the paint removed from the interstices and the inner wires examined. They were found to be in as good condition as when first put in.

The outer defective wires were cut away so as to uncover the second layer of wire at the bend of the shoe, when the second layer, or course, was found to be sound and bright.

Thus it was found that the only wires affected were the outer wires of the outside strands. Near the cylindrical portion of the cables, the outer wires were slightly rusted clear around the cable, but as we approached the shoes, the etching appeared to work toward the lower strands, till, when the shoes were reached, the principal corrosion was of the outer wires underneath the bottom shoes.

The evident cause of this corrosion was the elongation and contraction of the strands under the passing loads, which had loosened the cement from the outside strands, allowing moisture to work in and finally reach the lowest point. The portion of cement among the strands would go and come in a body with them.

Occasionally a limestone spawl had been carelessly left in contact with a strand, when they were being covered, in which case the contact was indicated by a black spot. On cleaning this away the wires immediately under it were found to be corroded partly through the outer layer.

While the examination was going on, the defective wires were cut out and new ones spliced in under strain. The greatest number of wires that required repairing at one end of any one cable was sixty-five.

On the 29th of March, 1877, Messrs. Milnor W. Roberts, T. E. Sickles and Col. W. H. Paine visited the bridge, at the request of the Board of Directors, and made an examination of the cables, both at the strands and at the saddles. These gentlemen reported that, in their opinion, as soon as the repairs, then going on, were completed, the cables would be in good condition.

A copy of their report was sent to the G. W. R'y Co., whose only reply was to call for a Commission of engineers to make an examination of the bridge and report upon its condition according to the terms of their contract.

The Bridge Companies then selected Col. Paine, the G. W. R'y Co. chose Mr. Thomas C. Clark, and these two gentlemen selected Mr. Charles Macdonald. The Commission arrived April 17th. During that and the three following days, they made a very thorough examination of the bridge and all portions of the anchorage that were accessible.

Very particular attention was paid to the condition of the anchor chains. Measurements of the sections of all the upper lengths of bars in each chain were taken, and of the sizes of heads and pins of the same, showing the following conditions:

1st. The outer bars of each set had a rather greater section than the intermediate ones, and the total section of the bodies of the nine links of a chain was 86.625 square inches, the average width of each bar being 7 inches, and the thickness $1\frac{3}{8}$ inches.

2d. The diameter of the pin was found to be scant $3\frac{1}{2}$ inches, or hardly five-tenths of the width of the body of the bar. The form of the eye bar head was approximately circular, with a diameter of about 12 inches. The centers of the pin holes were distant from the centers of the heads, and toward the body, so far that in some instances the minimum section on each side of the eye was less than half that of the body of the bar.

3d. Some of the pins through the shoes were found to be bent convex toward the cable 5-16 inch in their total bearing lengths.

4th. Two or three of the pin holes were open, back of the pin, nearly $\frac{1}{8}$ inch, while at each side of the pin they were close down, showing the holes to be eliptical. This was probably caused by the hole having been bored a little too large, which allowed them to elongate, or the bars may not have been quite long enough. At the close of the inspection a test load of nearly 450 tons, composed of a switch engine and twenty loaded box cars, was run upon the bridge and points of the curves of undulation taken with a level for each position of the load, at stations 100 feet apart.

The bridge regained its original camber when the load passed off. The Commission also made many tests of specimen wires cut from the strands.

After due consideration the Commission reported substantially as follows:

That the repairs of wires, affected by rust, having been completed, the action of the wire portion of the cables indicated that they were in good condition.

But regarding the anchor chains, it was believed that the

form of the heads and size of the pins would not enable these parts to transmit a greater ultimate strain upon the bodies of the links than 40,000 lbs. per square inch of their total section. Consequently while each of the cables possessed an ultimate strength of 3,640 x 1648=5,998,720 lbs., or say 6,000,000 lbs., the chain would not probably sustain a greater strain than 86.625 x 40,000=3,465,000 lbs. or 6,000,000—3,465,000= 2,535,000 lbs. less than the cable. The section of new chain necessary to supply the deficiency, estimated at 50,000 lbs. per square inch would consequently be 50.7 square inches.

The report was accompanied with plans for this reinforcement and required that it should be made.

The report also suggested the renewal of the suspended superstructure with iron, and submitted a general plan for that purpose.

REINFORCEMENT OF THE ANCHORAGE.

The plans accompanying the Commission's report were prepared from data obtained from Mr. Roebling's published report on Niagara Suspension Bridge. It proposed the sinking of one new pit into the rock at the back end of each anchor wall, with a chamber at the bottom for the reception of three small anchor plates. To the middle plate were to be attached two chains which were to pass vertically upward to the surface of the rock and thence to follow a curve till they became tangent to the line of the upper cable, and thence to pass one on each side of the old chain and attach to the strands of the cables. This connection was to be formed as follows: (see plate I.) Into each shoe was to be fitted a cast iron block with one end concave to fit against the pin. There were seven of these blocks at each end of a cable. The outer ends of all these blocks to be dressed off in the same plane at right angles to the center line of the cable at this point. Two cross bars, with a side of each planed, were to rest against the cast iron blocks and have their ends turned for the upper ends of the new chains to take hold of.

Each of the other two chains were to be made fast to one of the outer plates, and passing upward, in a curve, till tangent to

the line of the lower cable, and thence passing up, one each side of the old wall, to attach to the lower cable in the same manner as in the case of the upper cable chain.

The strains were to be applied by leaving one of the joints of each chain, on the curve, about two inches nearer the center of curvature, than any of the others and having the chain in this position just the right length to allow of connecting to the cable, then expand the chains by heat and as they lengthened raise the low joint and block it with iron plates, after which the cooling of the chain would cause it to contract and thus relieve the old chain of a portion of its load.

MEASUREMENT OF STRAINS.

Each bar of the upper length of each chain was to be subjected in a testing machine to a stress equal to the permanent load to come upon it and the corresponding elongation noted. The same elongation to be given to them when in position.

EXECUTION OF THE WORK.

The plan above described was to be subject to such alterations as circumstances should require.

I was selected as engineer in charge of the reinforcement and arrived at Suspension Bridge September 13th, 1877, and commenced at once the removal of the earth to uncover the rock.

On getting to the surface of the rock, it was found that the profiles of the anchorage given in the report of Mr. Roebling, from a typographical error, was a combination of the profiles of both sides of the river, that is, that the wall of the Canada anchorage and the height of the rock surface on the New York side were given in the same profile. Hence by placing the pits at the back of the old wall in each case, the line of the lower cable, on the Canada side, would enter the rock 30 feet away from the pit, and that on the New York side 12 feet. This would require expensive trenches in the rock, the cutting of which would necessitate blasting nearer to the old anchor pits than would be safe.

I accordingly made alterations in the plan which will be understood by referring to Fig 1, Plate I.

In this plan the pits were located the same as in the other, but smaller. One anchor plate in each pit was made to answer for all the four chains. There were eight links secured in the plate by one pin, and the first joint (*c*) was secured by one long pin. Beyond (*c*) each of the four chains was independent of the others, but had the same curvature and rested on the same stone supports. Two of the chains connected with the upper cables. The other two passed along grooves cut in each side of the wall, passing the supports of the old upper cable chains and fastened to the lower cable.

As will be seen by Fig. 1, Plate I, the plan followed, required a bend in the lower cable chain, to bring it on to the line of the cable. This was done by dividing the change of direction among three points, and securing them in position by means of stirrups attached to the ends of the pins of the old chain, as shown at *a*, *b* and *c*.

The connection with the strands was made as in the first plan, excepting that instead of the last links attaching directly to the cross bars, a triangular link is interposed, as shown by the dotted line in Fig. 2. The arrangement of the holes in this link is shown in Fig. 3. The object of the link is to cause a proper distribution of the strains. There being three strands above and four below, three-sevenths of the strain should go to the upper, and four-sevenths to the lower strands. Hence the resultant of the strain on the entire chain was made to coincide with the center line of the cable at this point. *e* and *f*, the centers of the upper and lower cross bars respectively, are in a plane at right angles to the center line of the cable, and the line joining *e* and *f* is intersected by the center line at a point, distant from *e*, four-sevenths of the whole distance, *a*. *d*, the point of attachment of the chain to the triangular link, is on the center line of the cable.

APPLICATION AND MEASUREMENT OF STRAIN.

These were made as before described in the case of the upper cable, but in that of the lower it was not necessary to apply heat, as the joint at A could be raised up and blocked, after which the strain was effected by screwing down the nuts

of stirrups *a*, *b* and *c* (Fig. 1) until the scale indicated the proper elongation of the eye-bars.

The total section of new chain attached to each cable is 50 inches for all that portion above the curve. From the upper end of the curve downward it gradually decreases till, at the anchor plate, it becomes 40 square inches.

METHOD OF DOING THE WORK.

1st. SINKING THE PITS.—In plan the pits are 6 ft. x 2 ft. 6 in. On the New York side they were sunk to a depth of 17 feet. On the Canada side to 23 feet. At the bottom the pits were chambered to 6 ft. x 7 ft. in plan, for the reception of the anchor plates.

In sinking the pits holes were first drilled along the four sides of each as near together as the drills would work without running the holes together, and to nearly the full depth of the pit. The core was then blasted out with light charges of dynamite.

The roofs of the chambers were dressed true and bush hammered. Just above the chamber notches were cut in the sides and ends of the pit.

2d. ANCHOR PLATES.—These are of cast iron 5 ft. 6 in. square and strongly ribbed. Each plate has eight cavities cored into it for the reception of the lower heads of the links, enclosing them perfectly. One pin passes through the whole eight links and all the partitions of the plate. The upper surface slopes outward and downard each way from the link openings.

After the plate was properly placed in the pit it was solidly concreted underneath. The stone blocks above the plate were cut to fit each place with thin joints, and the pieces as large as could be got into the chamber and notches. All vacant places were filled solidly with stone and cement, but no stone was permitted to come in contact with the chains.

In removing the portions of the walls necessary to place the new chains, they were taken down nearly to the positions for the beds of the knuckle supports, after which the beds were

prepared with points and bush hammer in order to give solid beds and thin joints.

The work of cutting the grooves each side of the walls for the reception of the lower cable chains required extreme caution, especially where they passed along the side of, and and partly under the ends of, the stone supports of the old upper cable chains. In the case of the south wall, on both sides of the river, these old supports had been solidly bedded, but in both north walls we found large cavities under two of them. In such cases the cutting had to be suspended till cement mortar could be forced under and have time to set, as of course the least settlement of any of these supports would destroy the adjustment of the cable if it did not endanger the bridge.

After the new chains were adjusted the masonry was rebuilt and both new and old chains covered and grouted solidly. But the wire strands were covered with brick houses. Each house is provided with a hatchway in the roof to give access to the strands for purposes of inspection and painting.

DURABILITY.

Possibly the question may be asked: Will these new chains continue to do their share of the work?

By referring to the drawing (Fig.1, Plate I) it will be seen that the only manner in which the chains can become slack, is from settlement of the knuckle supports and shrinkage of the joints around the anchor plates. But these joints are all thin and the mortar had become thoroughly set before the strain was applied.

At the time the strain was applied, the sun was shining directly upon the chains, making them warmer than they can possibly become again, covered with masonry as they now are, and hence the strain would naturally increase by covering them. Again one of the upper cable chains was left uncovered from D to F and without any support at E for two weeks. That joint settled only half an inch out of a right line and remained so during the two weeks, notwithstanding that the writer

jumped upon it and shook it in order to overcome the friction of the joint

In concluding this report on the anchorage, it is proper to speak of the developments made by uncovering the old chains.

From the condition of the cement around the curved portion of the chain it appears that the masonry covering them in the wall was put on before the weight of the bridge came upon them. As this weight was added the curved portion of the chain had settled forward toward the river and downward. This settlement was greatest near the top of the wall. From this point downward it decreased till about the third joint down where it was very slight. That such settlement had occurred was shown by there being cavities behind the chains and ends of the pins, both of which were rusty, while in front of the chains and pin ends the cement was close, and when chipped off left the iron bright as when new.

The rust was all scraped off and the old chains painted, after which they were re-grouted and covered by the masonry, so that no water can reach them again.

I neglected to mention, in its proper place, that the cross bars resting against the cast blocks in the shoes are of crucible steel, of a pretty high grade, but thoroughly annealed. There was not sufficient room between the strands to properly place iron bars of sufficient size to afford the proper strength and stiffness.

RENEWAL OF THE SUSPENDED SUPERSTRUCTURE.

While the work of reinforcing the anchorage was in progress, I made a careful study of the old superstructure, both to ascertain its condition and also the requirements of a new iron truss that would be adapted to take its place.

THE CONDITION OF THE OLD TRUSSES.—About they ear 1873, the upper floor had been renewed. The lower had, in addition to the auxiliary chords and slip beams (already mentioned), been repaired from time to time, new pieces being inserted, here and there, as the old ones had become decayed or broken.

As the strain from the trusses had to be transmitted to the auxiliary chords by the transverse stiffness of the floor beams, the frequent wrenching, to which these latter had been subjected, had opened their fibre, and, from the action of water and frost, had caused them to decay with excessive rapidity.

The old truss rod chord had become nearly worthless before the auxiliary one could act, and when the latter did begin to act it was compelled to do the whole work, consequently it had already parted in several places.

In short, while it appeared that by constant repairing the structure had been kept in safe condition, it was evident that the expense of keeping it so must greatly increase, and that, at no very distant date, it would become necessary to renew the entire truss system either in wood or iron.

Whatever course should be decided upon, it was necessary that the work should be done without stopping the traffic. To effect an entire renewal, with either wood or iron, without stopping the traffic, would require an entire change from the old plan. It was desirable to decrease the weight as much as possible. To do so with timber would require the best of sound, clear material, which is every year becoming more scarce and expensive. Even then it is doubtful whether a wooden structure, possessing the requisite strength, could be made of less weight than the old one on account of the loss of strength caused by cutting away material to form splices suitable to resist tensive stresses. Moreover the best constructed wooden structure would require renewal again in a few years.

The manufacture of iron had reached a degree of perfection that rendered it a cheap material, and it was open to none of the objections pertaining to wood.

CONDITIONS TO WHICH THE NEW SUPERSTRUCTURE MUST BE ADAPTED WHEN MADE OF IRON.

1st. The object of the trusses is to prevent too great undulations of floors and cables from the action of partial live loads and from wind, but yet it must not possess too great rigidity.

2d. The cables being anchored at both ends, there is, between extremes of temperature, a difference in versed sine of about two feet. The trusses must be arranged to accomodate themselves to that change without taxing the cables too much in deflecting them.

3d. If stays from the towers to different points of the floors continue to be used, the trusses should be continuous, and the middle point of their length should have as little movement endwise as possible in order to make the stays effective at all temperatures. The only use of the stays is to assist the trusses.

The first of the above conditions is met by giving to the trusses a depth such, that with the maximum deflection, allowable, under a maximum bending load, the members will not be subjected to more than the proper stress per square inch. Then give to each member such a section as will enable it to resist greater deflection.

The second requirement is satisfied in a manner similar to the first, that is by giving the trusses a depth that will enable the cables to bend them half the amonnt of their change of versed sine without too great stress upon cables or truss members Probably, in most cases, if this condition is satisfied, the depth will be found to be right for the first condition.

The third condition may be satisfied by making the trusses continuous from end to end, while at each abutment is arranged some means for automatically limiting the movement endwise to a given amount.

In the present case the depth of truss was approximately fixed by that of the old structure. But the depth was reduced as much as possible by placing the lower chord on top of the lower floor beams instead of below them as in the old structure.

While the work of reinforcing the anchorage was going on, I prepared plans for renewing the suspended superstructure in iron. They received Col. Roebling's approval, and it was my wish to let the contract for furnishing the materials so that it could be delivered during the fall and winter of 1878–9 and be ready for erection in the following spring. But the Board of Directors was not ready to commence upon it at that time and nothing more was then done about it.

During the following winter the question of using steel for the suspended superstructure of the East River Bridge came up and I turned my attention to obtaining what information I could regarding that material, to determine its adaptability to this work. I here take occasion to acknowledge my indebtedness to Col. W. H. Paine for many valuable notes and suggestions upon the use of steel for structural purposes.

There appeared to be no doubt as to its being the best material for many of the members of the bridge, provided that suitable shapes could be obtained, as its great strength would admit of decreasing the dead load of the bridge materially. But the use of steel had not yet reached a point at which all the required shapes could be obtained economically.

Consequently I prepared specifications which would be adapted to the use of either iron or steel, or a combination of both materials.

During the discussion of the steel question I submitted my general plans to the members of the Commission for their approval which was given in a letter to the Board of Directors.

On the 7th of May, 1879, a committee on construction, appointed by the Board of Directors, met at the bridge office, and after discussing the plans and specifications, authorized me to invite tenders, from manufacturing firms, for furnishing the materials and erecting the same.

May 28th the bids were opened in the presence of the Board and the contract awarded to the Pittsburgh Bridge Co., that company being the lowest bidder.

By the specifications and the terms of the contract the materials were all to be delivered at Suspension Bridge, ready for erection, by August 1st, 1879, and the erection completed by November 1st, 1879.

The specification gave a preference to Bessemer steel, made under the "Hay Process," for the reason that it had been the material used, exclusively, in a bridge built at Glasgow, Mo., by Wm. Suoy Smith, and which had given very good results. But the Pittsburgh Bridge Co.'s tender was for Bessemer steel and for iron. Bessemer steel met the requirements of the specifications very well as will be seen by the following extract from that document:

QUALITY OF MATERIALS REQUIRED.

All wrought iron must be of a tough, fibrous character, double refined. Bars, of moderate dimensions, must be capable of being bent cold, through 90°, with an inner diameter of curvature, not exceeding the thickness of the bar, without cracking. When nicked and bent sharply over the corner of an anvil, the fracture produced must show a good clean fibre. It must have an ultimate tensile strength of at least 50,000 lbs. per square inch of sectional area, and an elastic limit of at least 25,000 lbs. per square inch, when tested in specimens turned to cylindrical form, the turned portion of which has a length of at least ten diameters. It must elongate at least 15 per cent. before rupture. The area of action at point of fracture should reduce as nearly 25 per cent. of original area as possible, in order to be uniform in hardness. A uniform modulous is desirable.

The steel must be of a quality such, that in dimensions to be used, it shall have an ultimate tensile strength of not less than 70,000 lbs. per square inch of section, and an elastic limit of not less than 40,000 lbs. per square inch in tension, or in compression, when the specimens have a moderate length compared to their diameters.

Specimens bent cold through 90°, with an inner diameter of curvature not greater than one and a half diameters of the bar, must not show any cracks or flaws. When the specimens are tested in tension, with a strain of 40,000 lbs. per square inch of section, their power for resisting shocks may be tested by striking them smartly with a hammer while under that strain, when they must not crack.

The specimens must elongate at least ten per cent. of their original length before rupture, and their ruptured sections must be reduced at least twenty per cent. of their original sections. Uniformity in modulus and ductility is to be secured as far as possible.

On reaching Pittsburgh I made some tests of specimens of Bessemer steel. They gave very good results; but the only shapes to be obtained in steel were plates, angles and small channels.

I then decided to use steel for the posts, chords, track stringers and lateral rods. All other parts to be of iron.

As the chords must be constructed of plates and angles riveted together, the net section would necessarily be reduced to a greater extent than if channels were used, on account of the greater number of rivet holes required, hence there could be no great advantage in point of weight by using steel, unless of a higher quality than that called for by the specifications.

I then found at what increase of price per lb. the Hay steel, with an ultimate strength of 80,000 lbs. and elastic limit of 48,000 lbs., could be obtained and decided to use that material for the chords.

PROGRESS OF THE MANUFACTURE.

Immediately after the contract had been awarded for furnishing this material, there started up a demand for iron and steel that was unprecedented. The mills, throughout the country, had more orders than they could fill. Consequently when the time arrived, at which the materials should have been delivered at the bridge, the work at the shops had scarcely commenced, and very little of the raw materials had come from the mills. It soon became evident that nothing could be done about erection during the fall of 1879. However the work generally went forward in the shop when there was sufficient material on hand to work to advantage.

Every precaution was taken to do the work in a satisfactory manner.

Tests were made upon specimens cut from every blow, and then from specimens cut from the actual shapes from each blow for all the steel used, and with no other preparation. (See appendix.)

In building the chords, the angles were first laid out for punching. The holes were punched about $\frac{1}{8}$ inch too small. They were then clamped to the plates, in proper position, and a drill of proper size passed through both angle and plate, the angle being thus reamed and at same time serving as a templet for drilling the plate. This prevented any possibility of one

part bringing any stress upon another, as the holes perfectly agreed.

The rivets were all driven by pressure. They were of steel in all the steel parts, but of a lower grade than the pieces in which they were driven.

For further information upon riveting and tests see appendix.

ERECTION.

In March, of the present year, I returned to Suspension Bridge to make arrangements for erection.

An examination was made of the condition of the lower floor beams. From this examination it was evident that the beams were not in a suitable condition to sustain the extra weight that would come upon them while the new work was being erected, from the fact, that in addition to the natural deterioration they would require deep notches cut into them, thus weakening them still further.

Consequently I decided to put the new iron beams in, nearly throughout, before commencing the work of erection proper.

A commencement was made upon this April 13th, 1880, but it did not go forward rapidly until the 21st of that month. The work began at the middle and proceeded toward each end. As it went forward the floor was taken up for a width of about 3 feet on each side and a temporary wooden railing was put up on each side of the carriage way, leaving, for that purpose, a space of 9 feet in width.

As soon as an old beam was taken out an iron beam was put in its place, the suspenders attached to its ends, and the old chords and posts firmly secured to it.

This part of the work was done by May 13th. On the 31st of May, most of the material having arrived, we started on the work of erection. At that time we had stripped the old bridge of all superfluous material, as cornices, the rails forming the broad gauge track, &c., in all decreasing the weight about 80 tons.

METHOD PURSUED IN ERECTING THE NEW WORK.

Plate IV is given to show the respective positions of the old and new work.

The new superstructure is made as wide as would admit of its being placed between the rods of the two old trusses. The bottoms of the iron lower floor beams were placed at the same level as those of the old beams. The tops of the new upper beams were about an in inch below the tops of the old ones at the ends.

The steel upper chords are 3 inches narrower than the lower ones, in order that, by cutting away 3 inches in width from the inner edges of the wooden chords and notching the tops of the wooden floor beams, the steel chord could be placed in position.

The outer channel of the lower chord came in the position of the foot of the inner half of the old post.

The erection proceeded as follows:

1st. Blocks of pine 4 in. x 6 in. and of various lengths were prepared and tacked, one to the foot of each post, (the lower ends of the old posts consisted of blocks of red cedar.)

2d. Beginning at the middle of the bridge five lengths of steel lower chord were placed in position, on each side of the lower floor. The red cedar blocks being removed to make room for them, and as soon as a length of chord was placed, the new 4 in. x 6 in. blocks were inserted between the foot of the inner half of the post and top of the steel chord.

As the chords were placed their splices were riveted up. The rivets for this purpose are of steel from $\frac{3}{4}$ in. to $\frac{7}{8}$ in. diameter. They were driven while hot, with 8 to 10 lbs. sledges, which had the effect of upsetting the rivet and making it fill the hole nearly as well as by pressure.

3d. Beginning at the middle again, one part of each upper floor beam was cut off and the middle portion taken out to make room for the iron beams. These were inserted by passing them through between the old track stringer, turning them up on edge and placing them. As soon as the iron beam was in place a pair of steel posts were set up, the lower chord pins passed through the chord, the foot of the posts, and the eyes of the truss rods stubs. The heads of the two posts were then placed under the ends of the upper beam and temporarily bolted to its lower flanges. The beam was then wedged firmly between the parts of the old track stringer with wooden wedges.

4th. As soon as twelve of the new bents were placed, three lengths of upper steel chord were placed in position, on each side and bolted to the top flanges of the iron beams. Their splices were then riveted.

5th. The new truss rods were then inserted. They are in pairs, made of $1\frac{1}{8}$ in. square iron. Each rod is in two pieces. Each piece has an eye at one end and a thread cut on the other. The piece in the lower chord is short and has a left hand thread cut on it. The two parts are connected and adjusted by a sleeve nut. Each rod extends from the foot of one post to the top of the third post from it, and they incline each way.

In erecting, but one rod was put in each place at first in order to save weight.

In the above order the work proceeded each way from the middle of the bridge toward each end.

6th. When 150 feet of the new work was in place, the new chords were securely clamped to the old by means of oak and pine timber.

In the case of the upper chord the timber was notched to the pin ends, rivet heads, &c., at the side of the steel chord and resting on the top of the wooden chord. The upper side of the wooden chord and the under side of the timber were notched at intervals for the reception of oak keys. The timber was then clamped strongly to both old and new chords, and the keys inserted in the notches.

The old and new lower chords were clamped by fitting pieces of 4 in. x 16 in. pine to the lattice straps and rivet heads on the under side of the steel chord. It was keyed to the top of the auxiliary chord. The whole was then bolted together.

7th. As soon as the clamps were secured, the work of renewing the old material was commenced at the middle of the bridge, and followed that of erecting the new, generally, with an interval of about 75 feet each way.

The portion of the new work thus put in place weighed about 1,100 lbs. per running foot of bridge. Hence there were seldom over 90 tons of new material overlapping the old, but at the start, being in the middle, this was equivalent to about 150 tons distributed, or deducting the 80 tons, saved by strip-

ping the bridge, we have left 70 tons as the probable extra dead load upon it, but as the trains had at the outset been limited to 190 tons, it is not probable that the total weight of live and dead load ever exceeded that of ordinary usage.

While the above changes were being made, the work of replacing the lower floor was going forward each way from the middle. As the planking of the old floor was laid longitudinally, while that of new is laid transversely, there was necessarily a gap between them, which was bridged over by a raised platform, under which the change was made, and which was moved along as the work progressed.

When the work of making the above changes had reached the towers that of completing the riveting was carried through. It was all completed, except changing the track, by the 25th of August. Up to that time the traffic had not been interrupted.

10th. After the work of replacing the trusses and floors was completed, that of renewing the track began at the middle and proceeded each way at the rate of 30 feet per day, or of 60 feet total. This could have been done without interrupting traffic, but as the Great Western Railway Company was to do the work of removing the old material of the track and put on the new timber, they preferred to take an hour each day, when there was no passenger train and scarcely any freight to cross, and make the change of 60 feet at one time. This gave the remainder of the day to preparing more old material for removal, and for completing the riveting of the new.

The whole change was completed September 17th. Since the completion of the changes above described the time has been occupied in making side walks, hand rails, putting on the new roof, making new fastenings for the overfloor stays at the saddle, on the towers, and in painting.

The brick houses have also been built to cover the strands at the ends of the cables.

ADJUSTMENTS.

The work of adjusting the new structure has been carried forward as rapidly as circumstances would permit.

The operation was as follows:

1st. To Adjust the Camber.—It was desirable to make it as nearly an arc of a circle as possible. Consequently the truss rods were slackened in order to allow the cables to assume their natural curves. Levels at each 100 feet, in the length of the bridge, were taken to ascertain its profile while hanging naturally, then the true arc, with a proper versedsine for mean temperature was calculated, after which the difference between the true arc and the actual camber was found for each of the points. The suspenders were then screwed up or unscrewed according as the structure required raising or lowering till the proper camber was given. As this raising or lowering progressed it was necessary to keep the rods slackened.

2d. Stress on Suspenders.—This adjustment was made by means of a hydraulic weighing machine. A portable wooden frame was made to rest against the under sides of the beams. One end of the machine was secured to a screw passing through a cross beam of this frame while the other end of the machine was attached to the screw end of the suspender. The nuts on the screw ends were then slackened off. Then by working the screw at the other end of the machine the right stress was given to the suspender as indicated by the index.

The first time of going over them the machine was attached to one of the suspenders and the proper stress given to it. Then, leaving the machine in that position, the five or six sus-suspenders on either side of it were adjusted by springing them laterally with the hand, using the suspender with the machine on it, as the standard. After going over them in this way from one end to the other the machine was tried upon an occasional suspender to verify the stress. This required to be gone over three times to bring them to proper adjustment.

3d. Truss Rod Adjustment.—The truss rods required to be adjusted at about mean temperature, at which time there should be no stress upon any of them when the bridge is unloaded.

Consequently at about mean temperature each of a pair of rods was first screwed up to an equal bearing and then the sleeve nuts were turned backward just half of a revolution. This gives the trusses a little more flexibility, decreasing the

temperature stresses and, at the same time, does not allow of more than the proper amount of undulation under a partial load, as will be seen by referring to Plate VI.

4th. OVER FLOOR STAYS.—The intention being to have the stays merely assist the trusses when the bridge is loaded from one end to a distance of three or four hundred feet out, they were adjusted by running a train on to the proper position and then screwing each of the stays up to the proper stress.

PRESENT ARRANGEMENT OF THE BRIDGE.

The cables remain as they were with the exception of the anchorage which has already been described.

Plates II and III show the arrangement of the truss system. But the action of some of the parts will require explanation.

1st. THE END ADJUSTMENT.—As before stated, in a suspension bridge of this sort, in order to make the overfloor stays (or those from the tops of the towers to different points of the floors) effective, a continuous iron truss is required, the middle point of whose length shall be as nearly stationary as possible. The trusses in this case are continuous from end to end. In order to keep the middle from moving toward either end the automatic device shown at the end of the lower chord, Plates II and III, was designed. Its action is described as follows:

In the prolongation of the line of the lower chord is an abutment casting A (Plate III) firmly secured to the masonry of the arch. This casting receives the end thrust of the chord. There is one of these castings at each end of each lower chord.

A bent lever B has its fulcrum E secured to A. At the end (D) of the short arm of the lever is hinged one end of a $\frac{3}{4}$ in. diameter round rod R. This rod extends through the lower chord to the opposite side of the river, where its other end is secured to the abutment casting by a nut (n). At the end (F) of the long arm of B is suspended a cast iron wedge C, which is interposed between the end of the chord and of the abutment casting. The action of the device is as follows:

The change in length of the chord, between extremes of temperature, is about $8\frac{1}{2}$ inches. If the middle of the chord is stationary each end will consequently move $4\frac{1}{4}$ inches between

extremes. The rod R, which lies loosely in the chord but otherwise is independent of it, is a little longer than the chord and will change in length, between extremes, $8\frac{1}{2}$ inches, or double the movement of either end of the chord. Hence the other end of the rod being fast, the end D will move $8\frac{1}{2}$ inches carrying the end of the lever with it at the same time that the end of the chord moves $4\frac{1}{4}$ inches. Arm E F of the lever is three times the length of D E, hence F will move $25\frac{1}{2}$ inches, or six times as far as the end of the chord moves. Consequently the wedge C is made with an inclination 1 to 6 of its length. There is one of these wedges at each end of each lower chord. When the chord contracts the rod contracts in the same proportion and at the same time, thus bringing a thicker part of the wedge between the chord and abutment.

There is half an inch of space at each end for the chord to go and come in before bearing upon the wedge, an amount which is very nearly constant for all temperatures.

The long rods lying inside of the chord they both keep at nearly the same temperature with each other.

The wedge has two surfaces of friction, and hence its inclination of 1 to 6 is far within the angle of friction of cast iron. Hence no matter what the pressure of the chord, it brings no stress upon rod R except what is required to sustain the weight of the wedge.

Should the wedge ever get caught by the chord remaining against it, there being scarcely two hours, day or night, in which a train does not pass, as soon as a train rests on the other end of the bridge the wedge will be released.

Fig. 1, Plate VI is to show the action above described.

2d. Action of the Overfloor Stays.—Fig. 1, Plate VI is to assist in explaining this.

In case of a wooden truss, the points of attachment of the stays to the floor move in vertical lines as the temperature changes. Their change in length is not sufficient to compensate for the amount that the cables move the floor vertically. Hence, if they are adjusted properly for cold weather, they will become so tight in summer as to break and *vice versa*. In the case of an iron truss with a slip joint anywhere beyond the

attachment of the stays and with the end of the truss fixed at the tower, the point of attachment will move in a line *c d*, in which case its action is worse than in that of a wooden truss.

But in the case of a continuous truss with the middle kept stationary, the point of attachment moves in the line *a b*. In this way the stay is made to compensate very nearly for all changes of temperature.

3d. The ends of the trusses are anchored to prevent vertical and lateral movement only.

At the New York end this is effected by a hinged strut.

At the Canada end, where the surface of the rock is nearly at the level of the lower floor, a casting, anchored to the rock, is provided with slots, in which blocks on the ends of the end pins of the lower chord are free to move in the direction of the length of the bridge only.

4th. SUSPENDERS.—On account of the ends of the trusses being anchored to the rock the suspenders near the ends require to be left slack. It would be as well if they were left off, but as they do no harm they were kept on more for appearance sake than otherwise.

5th. UPPER FLOOR.—The deep transverse beams of the upper floor, together with the deep longitudinal track stringers, distributing the load over several beams, and the upper cable suspenders attaching to the beams, as shown in the transverse view, Plate II, form a very stiff floor. This makes the office of the knee braces that of steadying the bents merely.

6th. LOWER FLOOR.—The planking of the lower floor is laid crosswise of the bridge and in one thickness, to enable water to drain off more readily and the floor to dry out quickly.

The narrow foot-walks at each side serve the double purpose of a clean walk for pedestrians and to prevent the snow being blown from the carriage-way in winter, while, at the same time, they confine the portion loaded with snow to the necessary width for carriages to pass each other.

7th LATERAL BRACING.—As neither the upper or lower floor has any longitudinal planking to afford lateral stiffness, diagonal rods of steel were introduced to supply the deficiency.

The fifty-six wire rope river-stays are retained to resist the action of high winds and consequently the diagonal rods are merely for the purpose of keeping the intermediate points of the chords in line.

STRENGTH OF THE BRIDGE.

The anchorage, cables and towers are primarily the supporting members of the bridge as before. Hence the strength of these members is to be considered first, in the order observed above.

1st. THE STRENGTH OF THE ANCHORAGE.—This has already been discussed under the head of Reinforcement of Anchorage.

2d. THE CABLES.—The number of wires in each of the four cables is 3,640. The original, average, ultimate strength of each wire was 1,648 lbs. This gave, as the strength of one cable, 3,640 x 1,648=5,998,720 lbs.=2,989 tons, or for the four cables=2,999 x 4=11,996 tons in the direction of their length. There is no indication of any deterioration in their strength, but suppose the strength of the four cables to be 11,000 tons in the direction of their length, or 1,511 lbs. per wire.

The present total suspended weight of bridge, between the towers and including cables and stays, is 1,050 tons. Taking the maximum live load upon the bridge at one time as 350 tons it makes the total live and dead load=1,400 tons. The maximum stress upon the cables is at the top of the tower. Their stress at this point, in the direction of their length, is to the total vertical load as 1.78 is to 1. Hence we have $\frac{11000}{1.78}$=6,180 tons. The factor of safety is then $\frac{6180}{1400}$=4.41.

This factor of safety, in a tension member as long as one of these cables, and where the load, from the time it starts upon the bridge till it produces its maximum effect, is rarely less than one minute, is ample.

3d. THE TOWERS.—The maximum load imposed upon the top of one tower by the cables is 700 tons, and acting in a nearly vertical direction. The least section of the tower is just below the top and is 64 square feet. Hence the pressure per square foot, from the cable, is $\frac{700}{64}$=10.94 tons.

Trantwine gives, as the crushing load for limestone, 250 to 1,000 tons per square foot. Hence if we take 250 tons as the crushing load $\frac{250}{10.94}$—23, nearly, is the factor of safety. Or, if a factor of safety of 10 should be accepted, the number of square feet necessary at the top of the tower would—$\frac{7000}{250}$—28 square feet, or a square whose side is 5.29 ft. There is no other place at which the pressure per square foot is so great as at the point considered. Hence we could remove a thickness of 1 foot 4 inches on all four sides of the tower and still have it safe.

The stone, of which the towers are made, is limestone that was quarried near the bridge. It is very strong when used where it is not exposed to moisture and frost, as for the inside of a wall. But where exposed as in the faces of the towers, its surface disintegrates or "flakes off," and if neglected, would in time work in to a depth that would endanger the tower.

Painting has been resorted to as a protection but it soon dries and cracks. In any case it should not be put on unless the tower is dry as after several weeks of dry warm weather.

It might be well to try a coating of asphalte mixed with some material to keep it from cracking.

But, in any event, the towers should be attended to and, where necessary, new stones should be set in the faces.

STRENGTH OF OTHER PARTS.

Suspenders.—The suspenders have not been renewed. There are 628 of them to sustain a maximum load of 1,025 tons, or each one sustains 1.63 tons. In no case does the load exceed two tons. They are of $4\frac{1}{2}$ in. circumference wire rope possessing when new an ultimate strength of 30 tons, giving a factor of safety of at least 15. I have examined pieces of wire from several of those that we have cut and tested them for bending. There is no doubt of their strength being ample.

The upper cable suspenders are attached to the upper floor beams by means of U shaped stirrups made out of $1\frac{1}{8}$ in. round iron.

The lower suspenders attach directly to the projecting ends of the lower floor beams and close to the lower chord.

STAY FASTENINGS.—The stays are of wire rope, same size as the suspenders. The fastenings of the stays at the towers have been renewed in a manner to render the stays safe from wear.

The lower ends of the upper floor stays have permanent iron fastenings riveted to the iron beams and tied to the chords and track stringers by means of iron bars. The lower floor and river stays are attached to the lower chord pins.

In short it has been the intention to have all fastenings permanent, in order that adjustments once made will not be disturbed by any renewals of wood-work that may be necessary.

STRENGTH OF THE TRUSS SYSTEM.

The trusses have been designed for a maximum load of .8 of a ton per foot run, and a length of load of 534 feet, using the formulæ given by Rankine with a factor of safety of 5.

For calculations on truss stresses, see appendix.

RESISTANCE TO HIGH WIND.

For this purpose we have the inclination of the planes of the cables and the same number of wind stays as in the case of the old bridge, while there is less than six-tenths as great wind-surface. In a very high wind, blowing steadily, the bridge at the middle swings to the leeward 5 to 6 inches and remains there while the wind continues, but the motion is not felt when one is on the bridge. When in that position there is more of the weight of the structure thrown upon the upper windward cable and the lower leeward cable, while the other two are relieved of a like amount. I estimate the resistance produced by the inclination of the cable planes alone in that condition at 30 tons. There are, at the same time, the 28 river stays on each side. Those on the leeward side are relieved while the stress on those of the windward side is greatly increased. As this resistance is partly downward and partly toward the wind they not only increase the resistance offered by the cables but will safely afford a direct united resistance of 150 tons. The 8,000

square feet of wind surface, taking the pressure of the wind at 50 lbs. per square foot, gives 400,000 lbs = 200 tons pressure. Where the wind blows in gusts it does not appear to affect the bridge perceptibly.

None of the unusually high winds of this fall, though blowing directly across the bridge, affected it sufficiently to be felt by a person standing on it.

The strength of the trusses, together with the overflow and river stays, will effectually prevent any vertical undulation from the effect of wind.

WEIGHT OF THE NEW STRUCTURE COMPARED TO THAT OF THE OLD.

The weight of the wooden structure, at its completion, was estimated by Mr. John A. Roebling at 1,000 tons. But at the date of the inspection, there having been a large amount of timber added to it, it was estimated to weigh 1,130 tons.

When the work of replacing the lower floor beams was in progress I had one of them weighed and found that owing to the amount of water that it held it was very much heavier than it had been estimated. I also weighed other pieces of the bridge and from these made a new estimate with the following result:

Total suspended weight between the towers, - - -	Old bridge,	1,228	tons.
	New "	1,050	"
Difference in favor of new bridge, - -		178	"

It is possible that the estimate of 1.228 tons is somewhat in excess. But as the new bridge is now higher in the middle than the old one for the same temperature, notwithstanding that the middle suspenders have been lengthened over 3 inches since its completion, that would indicate a decrease of considerably over 100 tons.

SUGGESTIONS REGARDING THE CARE OF THE BRIDGE.

1st. As soon as dry weather comes next spring it would be well to remove the hatchway covers from anchorage houses,

and, if there should be any dampness, let them dry thoroughly and then paint all exposed iron work and wires. There is not much probability of dampness as there is ample ventilation at top and bottom, but the covers should be removed once a year and left off during one warm, dry day, and any part painted which appears to need it.

2d. The caps on top of the towers had better be removed at the same time, the parts painted and machine oil put in around the edges of the saddles.

An examination of the cables, from time to time, will enable an experienced man to determine when they require painting. They should be painted as often as the paint gets so hard and dry as to crack.

2d. The new work should be painted whenever it requires it Those portions near the intersections of chords, beams, posts, &c., will require painting oftener than other parts.

3d. Speed of Trains.—The durability of the bridge and (in case of derailment) the safety of the trains render it advisable that some more efficient means should be adopted for enforcing the article of agreement, limiting the speed of trains on the bridge to five miles per hour.

SUPERINTENDENCE.

A competent superintendent will, of course, continue to be necessary.

I trust that it will not be considered out of place for me to say here, that judging from my acquaintance with Mr. W. G. Swan, it is my opinion that no better man than he can be found to fill the position. He has at all times shown himself to be conscientious in the discharge of his duties, is acquainted with all the details of the bridge, and will therefore be competent to judge, by inspection, of what is required from time to time for its preservation.

In concluding this report I wish to express my sense of obligation to the Bridge Companies' Superintendent, Mr. W. G. Swan, for the willing assistance he has always rendered in promptly supplying me with necessary tools and materials, as well as for his friendly interest in the success of my work.

Also to the Pittsburgh Bridge Co. who, in supplying the iron and steel materials, called for by my plans, never spared any pains necessary to secure materials and workmanship of an entirely satisfactory nature.

Nor must I neglect to award great credit to my foreman, Mr. William Gardner, and to the workmen, who, by careful attention to orders, did so much toward enabling me to complete, and without an accident, a work that otherwise would have been dangerous.

In resigning my position as engineer of the work, I take pleasure in acknowledging to the Presidents and gentlemen of both Boards of Directors my indebtedness for their kindness toward me and for the unvarying confidence they have manifested in my professional judgment.

L. L. BUCK.

APPENDIX.

CALCULATION OF STRAINS UPON THE TRUSSES.

The formulæ given in Rankine's Applied Mechanics, for "Stiffened Suspension Bridges" have been principally the ones used in the following calculations, but with some modifications deduced from observation of the action of the old bridge under partial loads.

The formulæ in question, except in one instance, treat the stiffened suspension bridge as an inflexible structure, which, of course, cannot be the case in practice, though, by giving to the trusses greater depth in proportion to their length, we could approximate to the supposed condition. But it would be an unnecessary condition in any case, and as before explained, is not admissable in the one here considered.

The old truss system of the bridge was very flexible, deflecting under partial live loads, in some cases as much as 2 feet in a length of 500 feet. Yet the trains passed over it easily, and after twenty-five years of constant use, the cables give no indications of ill effects from the undulations.

In designing the new trusses I considered a maximum deflection of 15 inches in a length of 500 feet as not at all excessive. Fig. 2, Plate V, is given to illustrate the advantage that is gained by giving to the trusses considerable flexibility, aside from the relief it affords to the cables in bending the trusses at low temperature.

By allowing a deflection of 15 inches in a length of 500 feet it was estimated that the intensity of the live load, per running foot of bridge, and to be used in the formulæ, was reduced by .2 ton. In other words, if the trusses and cables were perfectly flexible it would require two-tenths of a ton per foot run to

produce the given deflection. The total intensity was taken at eight-tenths of a ton. Deducting the two-tenths of a ton we have six-tenths of a ton to use in the formulæ.

The formulæ are as follows:

For bending moment of trusses when the load begins at one end of the bridge and covers a given portion of it, we have for the unloaded segment—

$$M_c = \frac{w\,(c + x)\,(c - x)^2}{16\,c} \qquad (1)$$

The moment for the loaded segment is—

$$M_a = \frac{w\,(c + x)^2\,(c - x)}{16\,c} \qquad (2)$$

$$\text{Max. } M_c = \text{Max. } M_a = \frac{2\,w\,c^2}{27} \qquad (3)$$

For shearing with the load as above we have—

$$F = \frac{w\,(c^2 - x^2)}{4\,c} \qquad (4)$$

$$\text{Max. } F = \frac{w\,c}{4} \qquad (5)$$

For deflection of the loaded segment when the load covers two-thirds of the span beginning at one end—

$$V_a = \frac{5}{27} \cdot \frac{f\,c^2}{E\,y} \qquad (6)$$

The formula for a distributed load, covering the whole span and required to bend the two trusses 12 inches, is taken from Moseley's Mechanics and is—

$$W = \frac{D \times 48\,E\,I}{5\,c^3} \qquad (7)$$

For chord strain produced by W, we have—

$$S = \frac{W\,l}{8\,d\,a} = \text{strain per square inch.} \qquad (8)$$

In the above formulæ.

W = intensity of live load per foot run = .6 tons.
c = half span of truss = 400 feet.
X = distance from middle of span to end of load.
f = strain per square inch, on chord in lbs.
c = half span of truss in inches = 4,800 inches.
E = modulus of elasticity of steel = 28,000,000.
y = half depth of truss in inches = 105.5 inches.

D = deflection at middle of truss from temperature = 12 inches.
I = Moment of Inertia of two trusses = 1,113,000.
l = length of truss in feet = 800.
d = depth of truss in feet = 17.58.
a = area of section of two chords = 50 square inches.

LOAD STRAINS ON TRUSSES.

From (3)

$$\text{Max. } M = \frac{2 \times .6 \times \overline{400}^2}{27} = 7{,}111 \text{ tons.} \qquad (a)$$

Hence strain per square inch on chord section =

$$S = \frac{7111}{17.58 \times 50} = 8.09 \text{ tons} = 16{,}180 \text{ lbs.} \qquad (b)$$

$$V_a = \frac{5}{27} \cdot \frac{16{,}180 \times 230 \times 23.040{,}000}{28{,}000{,}000 \times 105.5} = 23.38 \text{ inches.} \qquad (c)$$

$$F = \frac{.6\,(\overline{400}^2 - \overline{134}^2)}{1{,}600} = 53.26 \text{ tons} = 106{,}520 \text{ lbs.} \qquad (d)$$

There may be two cases in which *a*, *b*, *c* and *d* will occur:

1st. When the load, beginning at one end of the span, covers two-thirds of its length, in which case S is at the sections of the chords at the middle of the loaded segment—the lower chord being in tension and the upper in compression. At same time (*c*) is the deflection of the middle point of the loaded segment, while (*d*) is at the ends of the loaded segment. *a*, *c* and *d* all act downward.

2d. When the load begins at the end of the span and covers *one* third of its length, the effect upon the unloaded segment is the same as in the first case upon the loaded segment, except (*a*), (*c*) and (*d*) all act upward and (*b*) is tension on the upper chord and compression on the lower.

$$\text{Max. } F = \frac{.6 \times 400}{4} = 60 \text{ tons} = 120{,}000 \text{ lbs.} \qquad (e)$$

e occurs when one-half the span is loaded beginning at one end of the bridge. It acts downward at the extremities of the loaded segment, and upward at the extremities of the unloaded segment.

STRAIN DUE TO TEMPERATURE.

At mean temperature there is no strain upon the parts of the trusses when unloaded.

But in winter the cables rising 12 inches lift the trusses the same amount while in summer their own weight bends them downward 12 inches. In both cases it is a question of bending a girder 12 inches by a distributed load of uniform intensity. Then by (7)

$$W=\frac{12\times 48\times 28,000,000\times 1,113,000}{5\times 110,592,000,000}=32,462 \text{ lbs.} \qquad (g)$$

From (8)

$$S=\frac{32,462\times 800}{8\times 17.58\times 50}=3,692 \text{ lbs.} \qquad (h)$$

(*g*) acts upward in winter, producing a shearing strain which is greatest at the ends of the bridge and equals—

$$\frac{W}{2}=\frac{32,462}{2}=16,231 \text{ lbs.} \qquad (i)$$

(*g*) acts downward in summer with the same force at the ends of the bridge$=\frac{32,462}{2}$ lbs.$=16,231$ lbs.

(*h*) acts on the chords at the middle of their length. In winter it is compression on the lower and tension on the upper.

In summer it acts at the same points but reverses the direction of the strains on the chords. The strain on the chords, due to temperature, diminish from the middle toward each end as the ordinates of a parabola whose parameter$=34,077$. Hence from the equation of the parabola.

$$y^2=2\ p\ x \text{ in which } x=134 \text{ feet.}$$

$$y^2=34,077\text{x}134=4,556,318,\ y=2,137 \text{ lbs.} \qquad (k)$$

(*k*) gives the stress, per square inch, on the chords at a point 267 feet from the end of the truss, or where the maximum stress from the load comes.

Examining the preceding values we find as follows:

1st. That to get the greatest strain per square inch on the chords, which will occur at either extreme of temperature, we add (*b*) and (*k*) and have $16,180+2,137=18,317$ lbs. (*l*)

In winter with the load one-third as long as the bridge beginning at one end, *l* is at the middle of the unloaded seg-

ment, is tension on the upper chord and compression on the lower.

In summer, with the load two-thirds as long as the bridge, l is at the middle of the loaded segment, is tension on the lower chord and compression on the upper.

2d. The greatest shearing force occurs at the ends and is found by adding (*e*) and (*i*)$=120,000+16,231=136,231$ lbs. (*m*).

In winter this occurs at one end of the bridge while the load is at the other end and acts upward.

In summer it is at the end of the loaded segment and acts downward.

This stress is resisted by six posts, each composed of two 6 inch channels, each having a section of 2.1 square inches, or the total section of 6 posts $=25.2$ square inches. Consequently the strain per square inch $=\frac{136,231}{25.2}=5,406$ lbs.

The stress (*m*) is also resisted by 12 truss rods whose united cross section $=15$ square inches, the tangent of whose inclination to the vertical is 1.31. Hence $\frac{136,231\times 1.31}{15}=11,898$ lbs. per square inch.

The anchorage for holding the ends of the span are secured by eight Lewis bolts, at each end of the bridge, $1\frac{5}{8}$ inches in diameter and sunk into the rock to a depth of 5 feet.

As before remarked the above calculations suppose a deflection of 15 inches at the middle of the loaded segment. But the formula gives it as over 28 inches which would decrease the intensity still more and hence reduce all the quantities given above.

The stays will also diminish the strains on the truss members especially those subjected to the shearing force at the ends.

NOTES ON THE TEST OF THE NEW WORK.

The actual deflections, under a load consisting of an engine, tender and thirteen box cars loaded, having a total weight of 357 tons, are given for eight different positions of the load by Figs. 2 to 9 inclusive (Plate VI). The total length of the train was 465 feet. In the diagrams the vertical scale is 12.5 times as great as the horizontal scale. Fig. 2, Plate V, is constructed from Fig. 5, Plate VI.

By applying the parallelogram of forces to the two segments of the cable (Fig. 2, Plate V), it is found that with the load in the position shown, covering a little over half the span, the intensity of the dead load of the unloaded segment balances not only the intensity of the dead load of the loaded segment, but .12 ton of the intensity of the live load beside. The total intensity of the live load$=\frac{357}{465}=.77$ ton. Consequently (w) in the formulæ$=.77-12=.65$ ton. Substituting this in (2) we get—

$$M_a=\frac{.65\,(\overline{400}+\overline{15})^2\,(400-15)}{6400}=6{,}733 \text{ tons.}$$

Strain per square in. on chords$=\frac{6733}{17.58\times 50}=7.66$ tons$=15{,}320$ lbs.

$$F=\frac{.65\,(400^2-15^2)}{4\times 400}=64.9 \text{ tons}=129{,}800 \text{ lbs.}$$

Hence, strain per square inch on posts=5,151 lbs.
" " " " " truss rods=11,336 lbs.

REMARKS.

The train considered in the above calculation is about the heaviest that will have occasion to cross the bridge and is about as heavy as can be started by an engine on the up grade of the New York side.

The overfloor stays will afford a very considerable assistance to the trusses, enough at least to compensate for the increased stress due to temperature. An experiment with a Vernier scale, reading to $\frac{1}{10000}$ foot, applied to the lower chord at a point 200 feet from one end to determine the elongation of 21 feet of its length, under a load of one engine and ten loaded box cars weighing about 280 tons, resulted in showing an elongation corresponding to less than 10,000 lbs. per square inch. The stays were not acting at that time.

The full and dotted lines at ends of Fig. 7, Plate VI coincide very exactly, apparently indicating that with the load in the position shown, the trusses are sufficiently rigid to distribute the load over the cables properly and consequently to produce no more effect upon them than if the load of same total weight was lengthened so as to cover the whole span. Figures 5 and 9, Plate VI, indicate a pretty exact adjustment of stays, suspenders and truss rods.

REMARKS ON TESTS OF STEEL.

The tests of specimens of steel for the suspended superstructure were made on the testing machine of Mr. W. L. Gill, manufacturer of car wheels in Allegheny, Pa.

The strains are given by a screw which enables the operator to stop at any stage of a test and yet be sure of the stress remaining constant. The strains are indicated by a beam scale. Elongations are measured by means of a michrometer with two screws reading to $\frac{1}{10000}$ inch. Contact of screws was indicated by a battery being connected and striking a bell.

It was a very satisfactory machine to work with.

Enough of the tests are given to show the action of the specimens under severe treatment. (See table of tests.)

No. 4 was cut from a plate by a planer. The tool raised a "lip" on the corner. In such cases, shortly after the specimen began to stretch, a nick would appear in the "lip" and when the specimen broke it would generally be at that point. It would break by tearing apart.

Nos. 5, 6, 7 and 8 were all cut from the same plate to test the effect of punching and reaming and also of annealing. Regarding 5 and 6 it appears a little remarkable that the punched and reamed specimen shows a less elastic limit but a larger ultimate than the plain specimen. I account for the less elastic limit by supposing that the stress was greater on one side of the hole than on the other, hence causing stretch to begin on that side first. The cause for the larger ultimate was no doubt due to the hole having the same effect that a semicircular groove does in a specimen, viz., by preventing reduction fracture must take place simultaneously over the whole section instead of tearing.

The effect of annealing appeared to be the same in nearly all cases, viz., to increase not only the stretch and reduction but also the elastic limit and ultimate.

No. 13 was cut from the same bar as 12. It was nicked on one side with a sharp chisel and when under a strain, considerably above the elastic limit, it was struck smartly with a hand hammer on the side opposite the nick, but showed no weakness. In breaking, the break started at the nick and gradually made its way through as in soft iron.

Nos. 14 and 15 were for the purpose of showing the effect of punching without reaming compared to that of punching *and* reaming. Calling elastic limit and ultimate of the punched and reamed specimen 100, that of the punched was for elastic limit 93, ultimate 84.

No. 16 was nicked across one side and after determining the elastic limit it was subjected to a strain of 59,370 lbs. per square inch, and while the strain was on it was struck on the side opposite the nick sufficiently hard to bend it $\frac{1}{4}$ inch and it retained a bend of $\frac{1}{8}$ inch with the strain on it.

Nos. 17 and 17′ were the same specimen. The elastic limit and modulus were first determined. It was then pulled to the maximum after which the strain was removed. The specimen measured for length and cross section, and then treating as a new specimen, it was tested for elastic limit, modulus and ultimate. The reduction given is the total reduction. The modulus is about the same for both tests. The object of this test was to determine if, in case one part of a bar of this grade of steel should receive a greater stress than the other parts, even to causing it to pass the elastic limit, its modulus would still enable it to resist in proportion to its section as much as before. If its modulus is not altered by stretching, the structure of which it formed a part might not be immediately rendered unsafe, but it is more a matter of curiosity than otherwise as nobody would think of straining material to that extent.

From a study of the tests of steel it appears that below the elastic limit, for a direct tensile stress, a slight nick would not be dangerous, but that the danger from the nick lies more in having transverse vibrations take place which, concentrating a heavy stress at the bottom of the nick, would cause it to break.

The tensive strain that was on the specimens at the time of striking them was no doubt an assistance in preventing transverse vibration in the bar, causing it to resist the effect of the blow, where the same blow would have broken it without the tension.

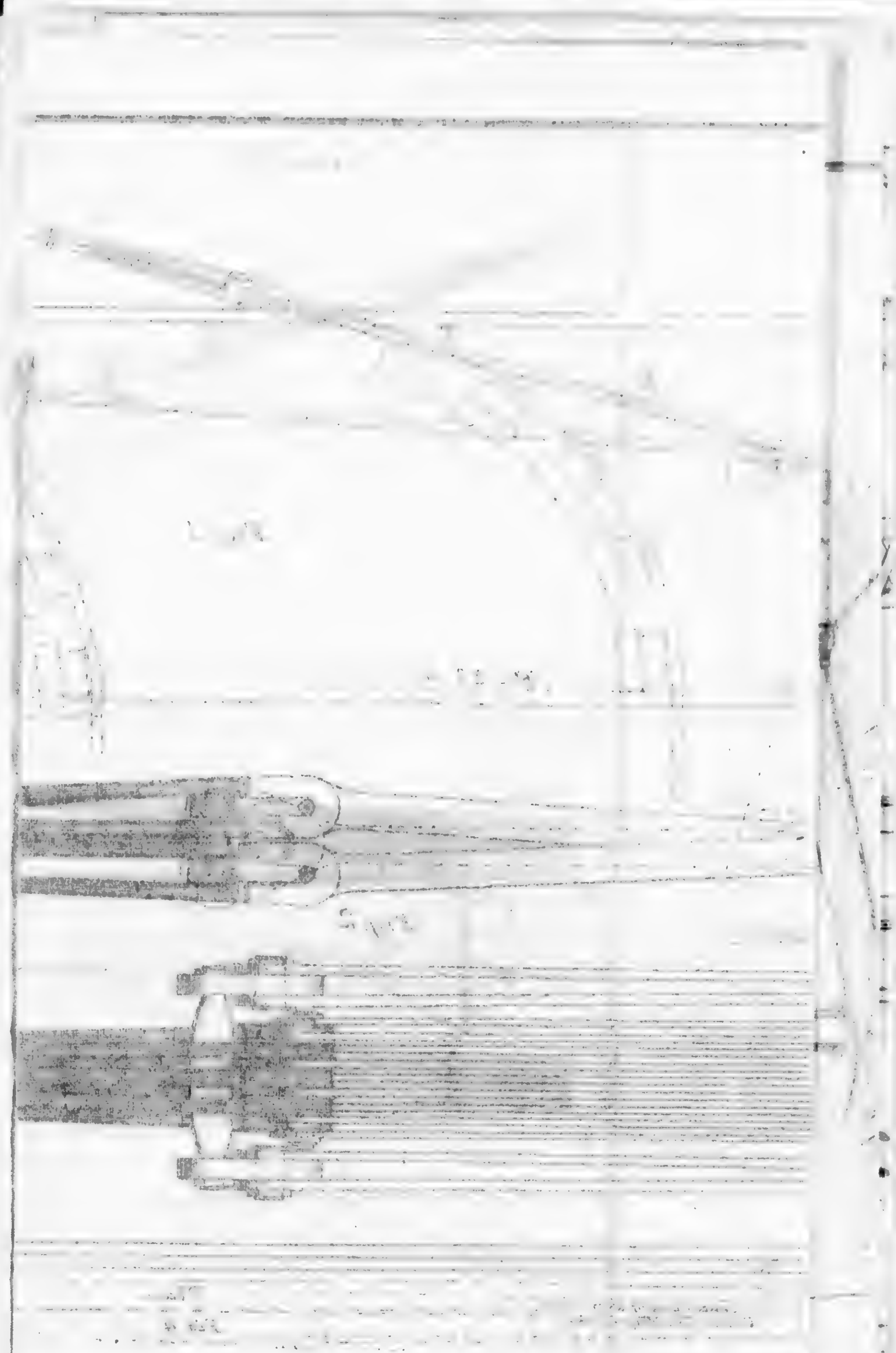
The modulus of steel, being so much less than that of iron in proportion to their elastic limits, gives to the trusses greater flexibility for the same depth of truss when made of steel than if made of iron.

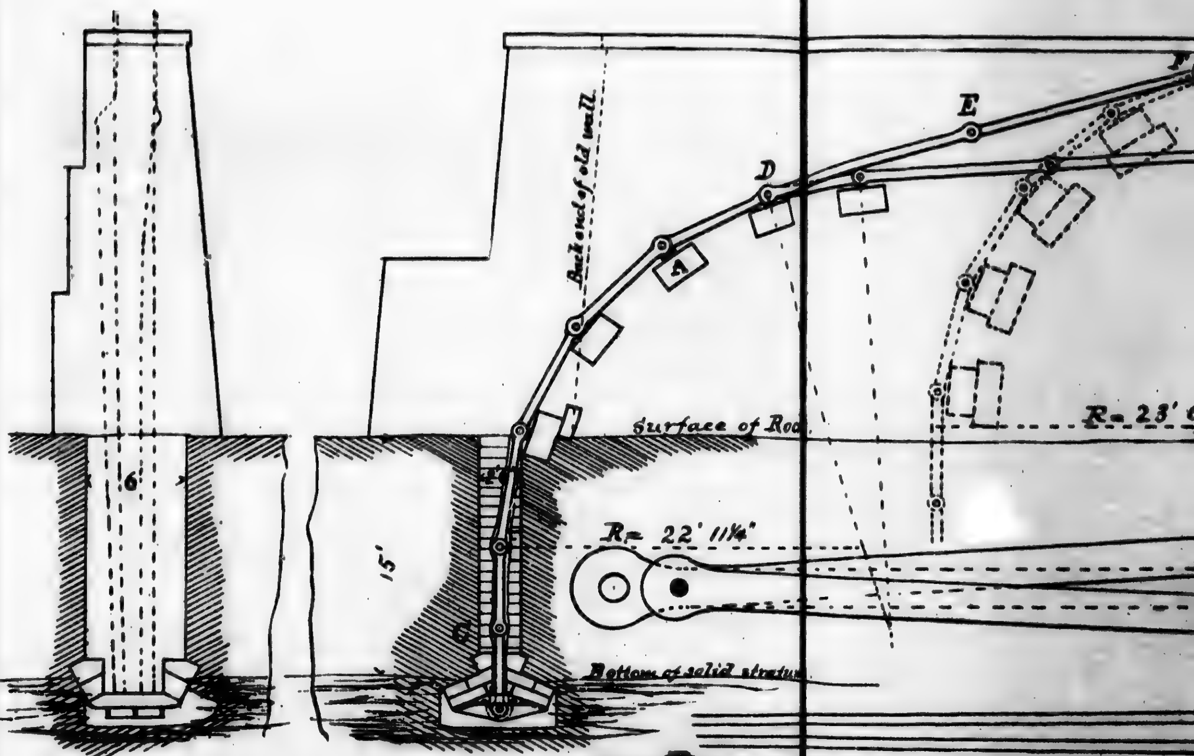
L. L. BUCK.

TABLE, SHOWING RESULTS OF TESTS OF SPECIMENS OF STEEL CUT FROM PLATES AND ANGLES FOR THE SUSPENDED SUPERSTRUCTURE OF NIAGARA BRIDGE. TESTS ALL FOR TENSION.

No.	Length between Clamps. Inches.	Cross Section. Dimensions. Inches.	Cross Section. Area.	Elasticity. Limit lbs. per sq. inch.	Elasticity. Modulus.	Strain. Maximum lbs. per sq. in.	Strain. At rupture lbs. per sq. in.	Percentage of stretch in 10 inches of length.	Percentage of reduction at ruptured section.	Form of Section.	Remarks.
1	10	1.228×.319	.395	54,430	27,400,000	84,050	82,300	15.6	23.8	Rectangular.	Cut from plate: bent to 1 dia. No crack
2	10	1.228×.315	.387	59,431	28,700,000	85,271	81,800	14.6	33.	"	" " " " " 1 " " "
3	10	1.483×.240	.356	54,775	....	82,275	74,000	19.4	42.	"	Lattice bar.
4	10	1.238×.382	473	56,000	28,700,000	96,540	95,434	12.	21.	"	Cut from plate. Lip on corner.
5	9	1.256×.433	.544	52,390	28,000,000	94,800	93,000	....	31.5	"	
6	8	2.263×.433	.625	47,200	....	97,600	....	In dia. hole. 13.7	14.	"	Punched & reamed. } 5, 6, 7 & 8, all
7	7	1.216×.429	.522	51,725	26,500,000	89,846	....	12	14.4	"	Unannealed. } cut from same
8	7	1.216×.429	.522	58,429	26,400,000	93,680	....	14.4	16.6	"	Annealed. } plate, rejected.
9	10	1.014×.319	.322	51,240	27,740,000	86,180	84,000	16.	34.4	"	Cut from plate.
10	10	1.205×.326	.373	52,160	28,000,000	86,300	86,000	15.	19.2	"	Bent to 1 dia., slight check in lip.
11	10	.747×.751	.561	48,100	30,500,000	87.300	85,000	17.	35.	Square.	Siemens Martin.
12	10	.747×.744	.556	50,360	30,830,000	74,000	....	17.	39.	"	" "
13	10	.747×.744	.556	51,250	....	78.000	....	8.75	16.	"	Off same bar as 12. Nicked and struck on side opposite nick at 61,000 lbs.
14	8	.495×1.451	.718	46,700	....	81,580	....	18.	16.	Rectangle.	Punched and reamed. Call result 100.
15	8	.495×1.421	.703	43,300	....	69,000	....	7.	13.	"	Same plate as 14. Punched as compared to punched and reamed. E. L.=93 max.=84.
16	10	1.265×.506	.640	53,125	....	59,370	....	....	....	....	Nicked, strained to E.L. struck, strained to 59.300, struck several times.
17	9	1.05 ×.375	.394	55,840	27,600,000	90,100	....	15.6	at 90.100	per sq. inch.	Cut from plate and pulled to E. L.,
17'	11	.973×.353	.342	90,640	27,500,000	106,100	105,000	....	21.	Rectangle.	then to max., then measured section and length and tested for modulus again as in 17. (See App.)

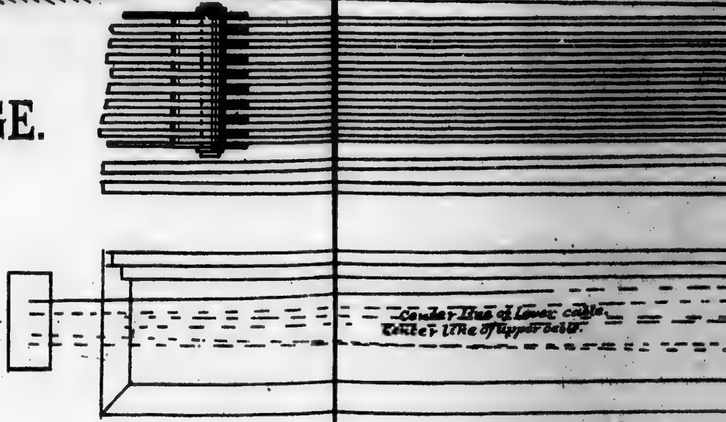
Average Modulus, 28,000,000.





NIAGARA
SUSPENSION BRIDGE.

REINFORCEMENT
OF
ANCHORAGE.



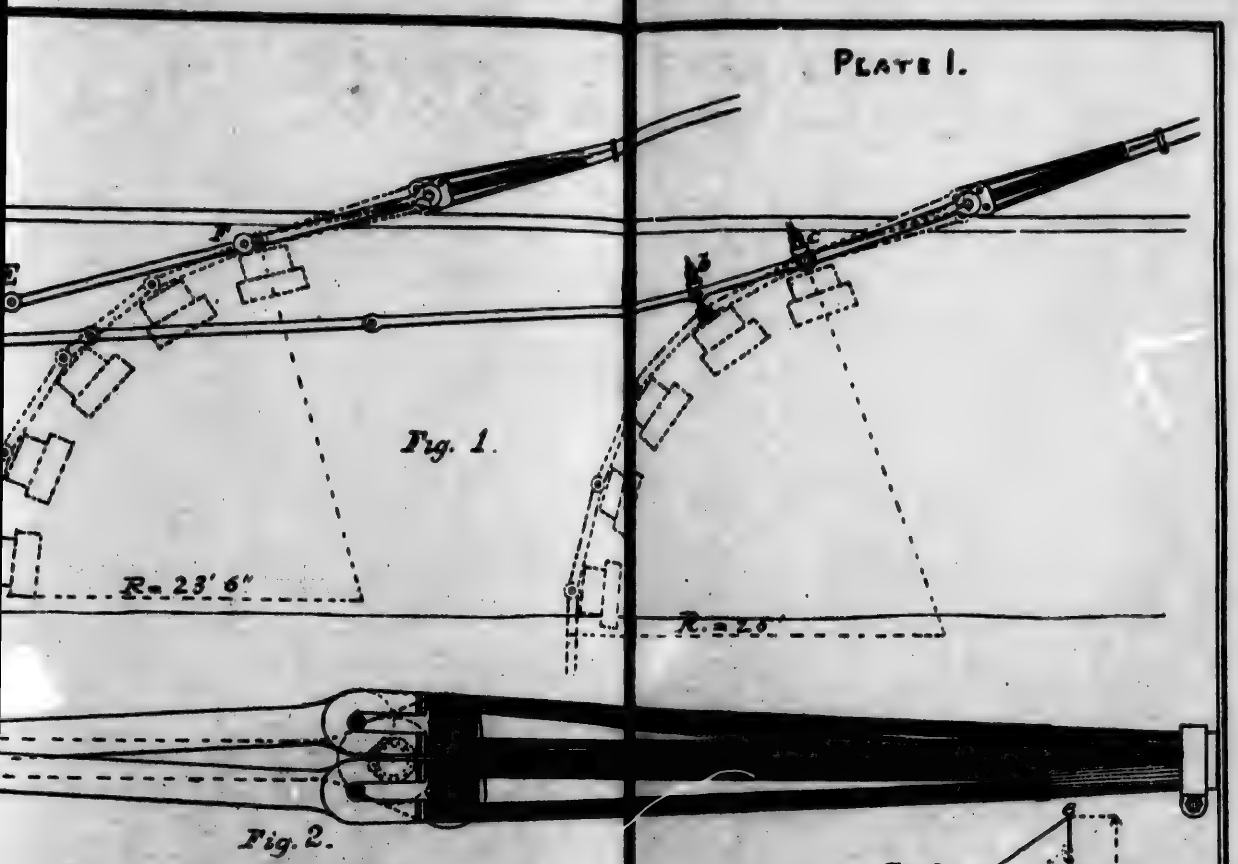
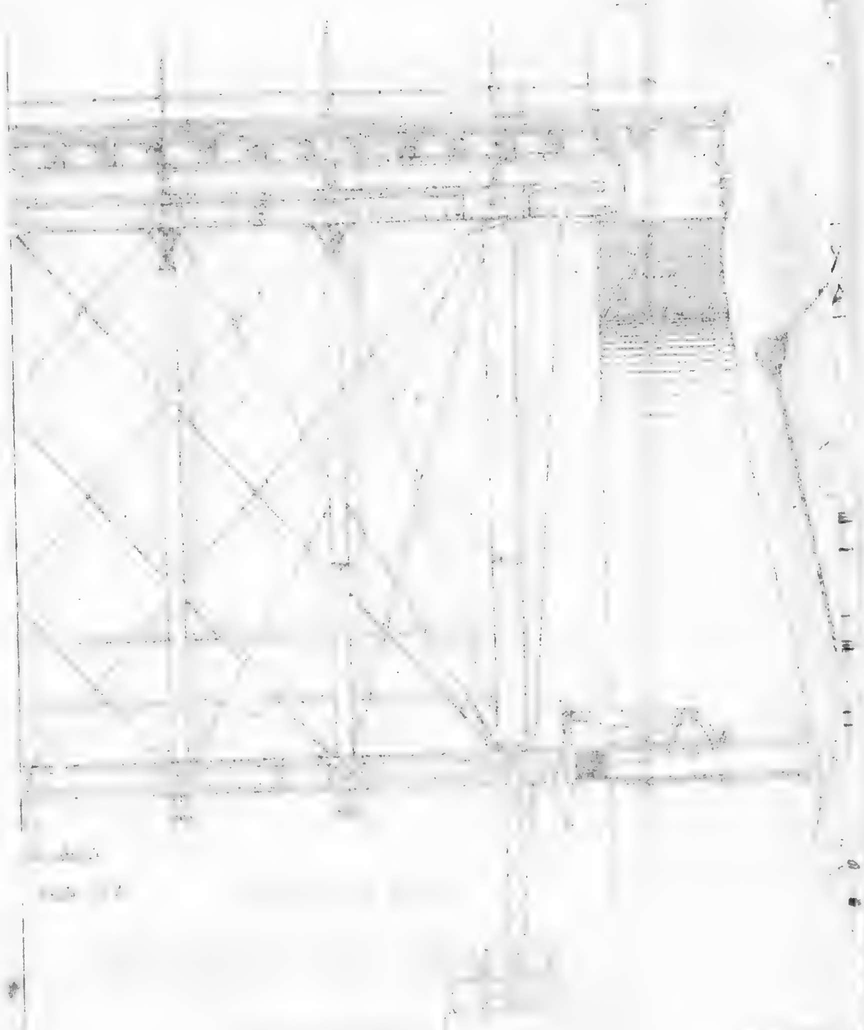


PLATE I.
Fig. 1.
R = 23' 6"
R = 20'
Fig. 2.
Fig. 3
Plan.
Fig. 4.



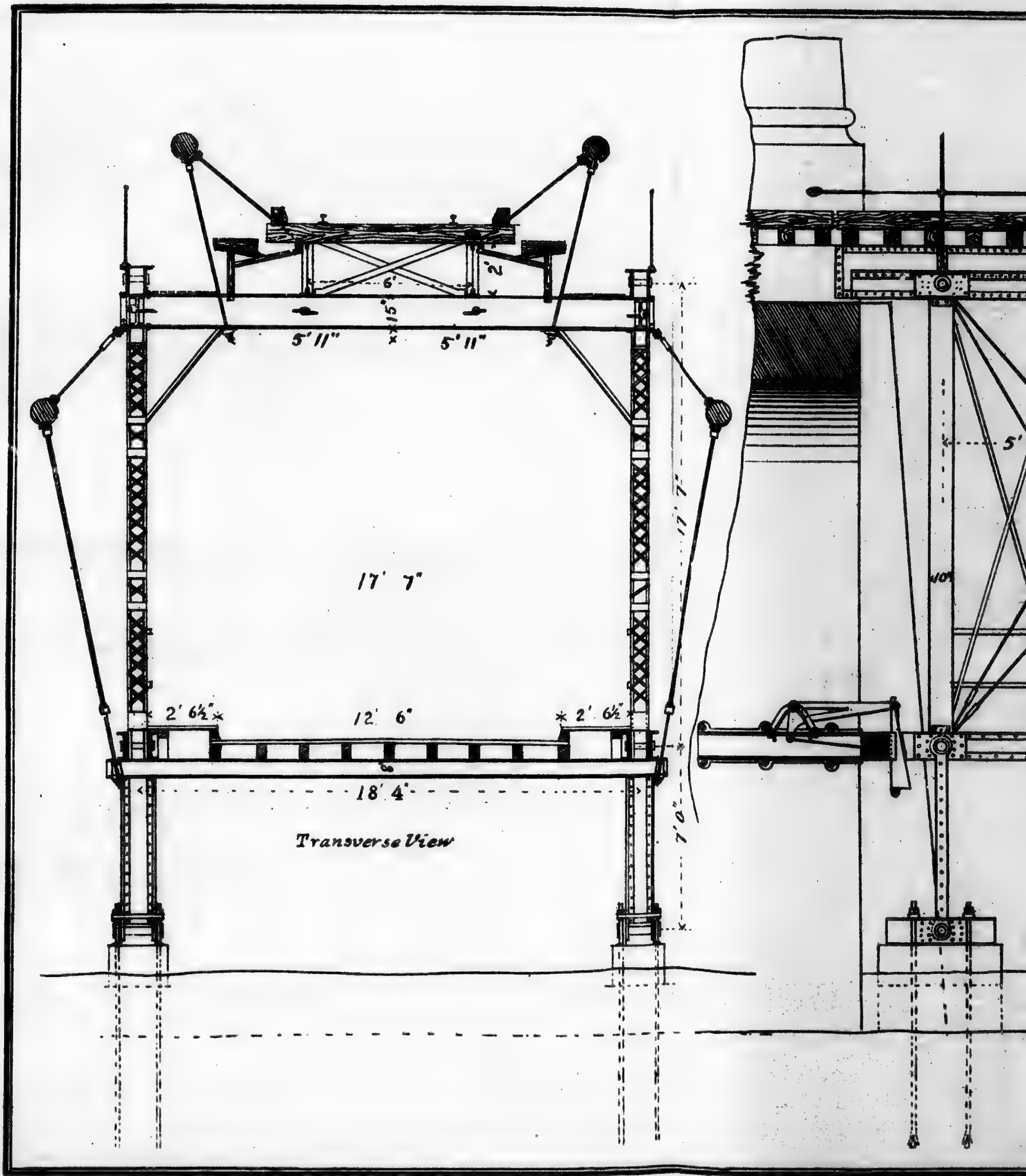
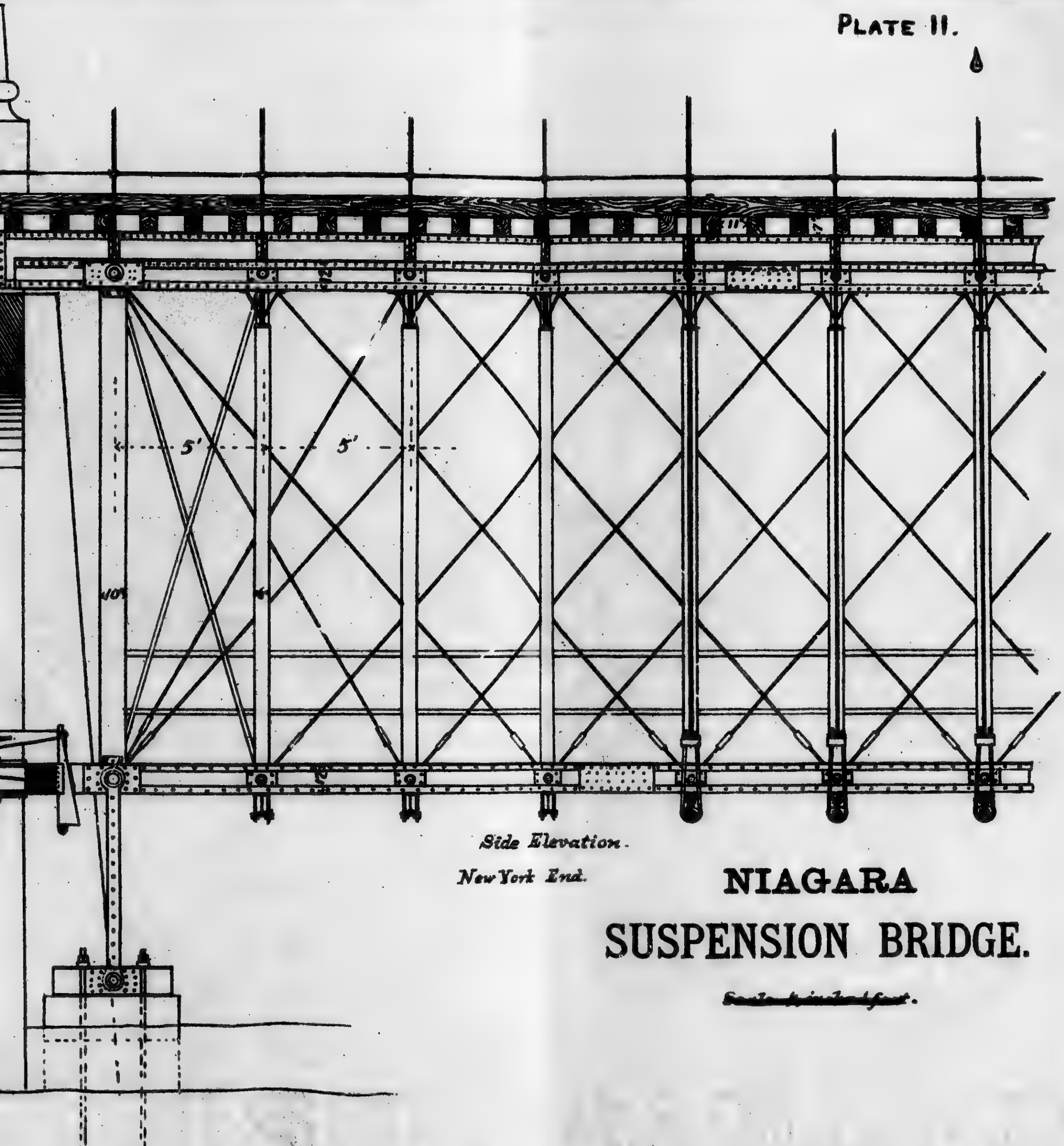

6'
2'
15"
5' 11"
5' 11"
17' 7"
17' 7"
2' 6½"
12' 6"
2' 6½"
18' 4"
7' 0"
Transverse View
5'
10"

PLATE II.

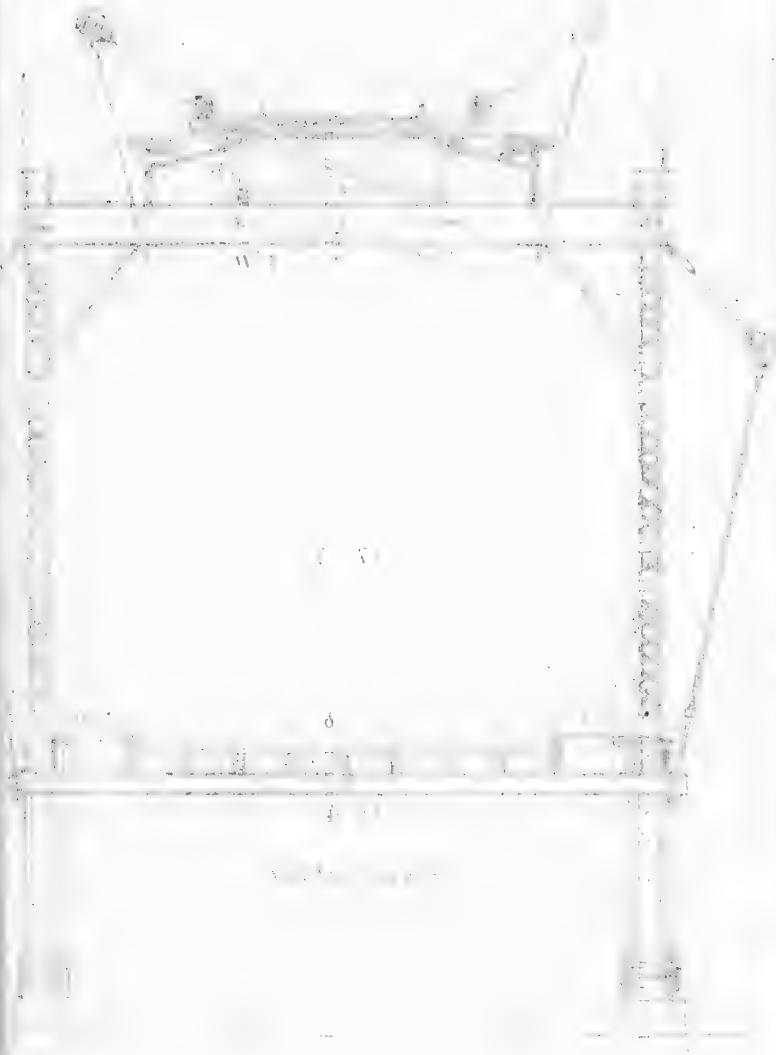


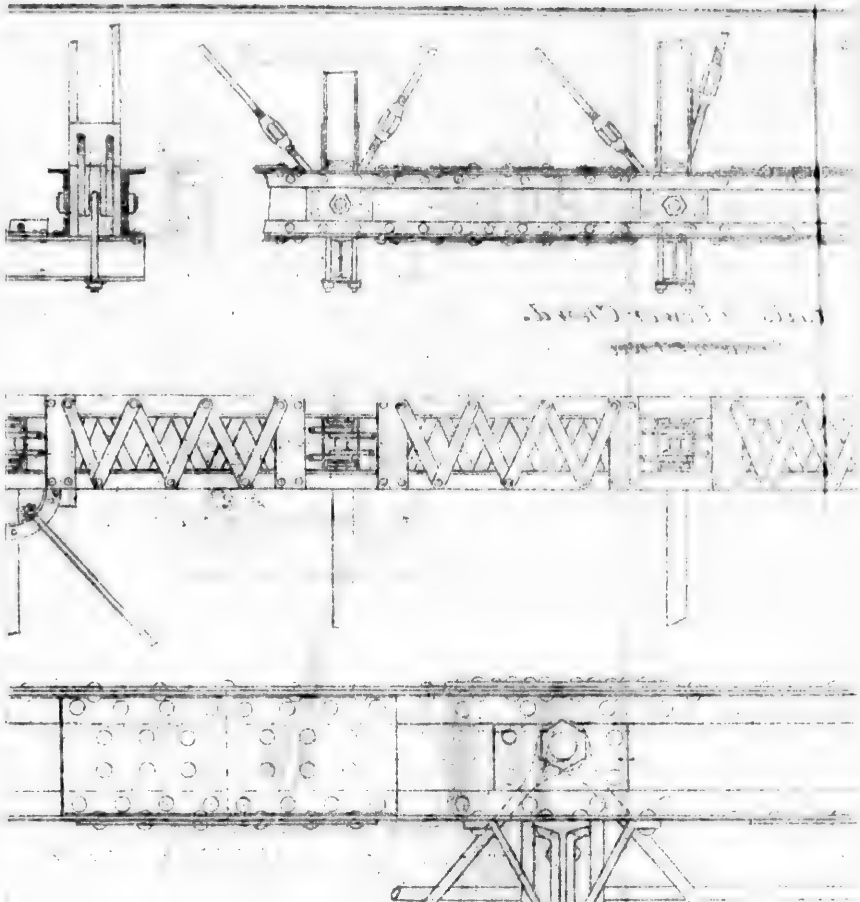
Side Elevation.

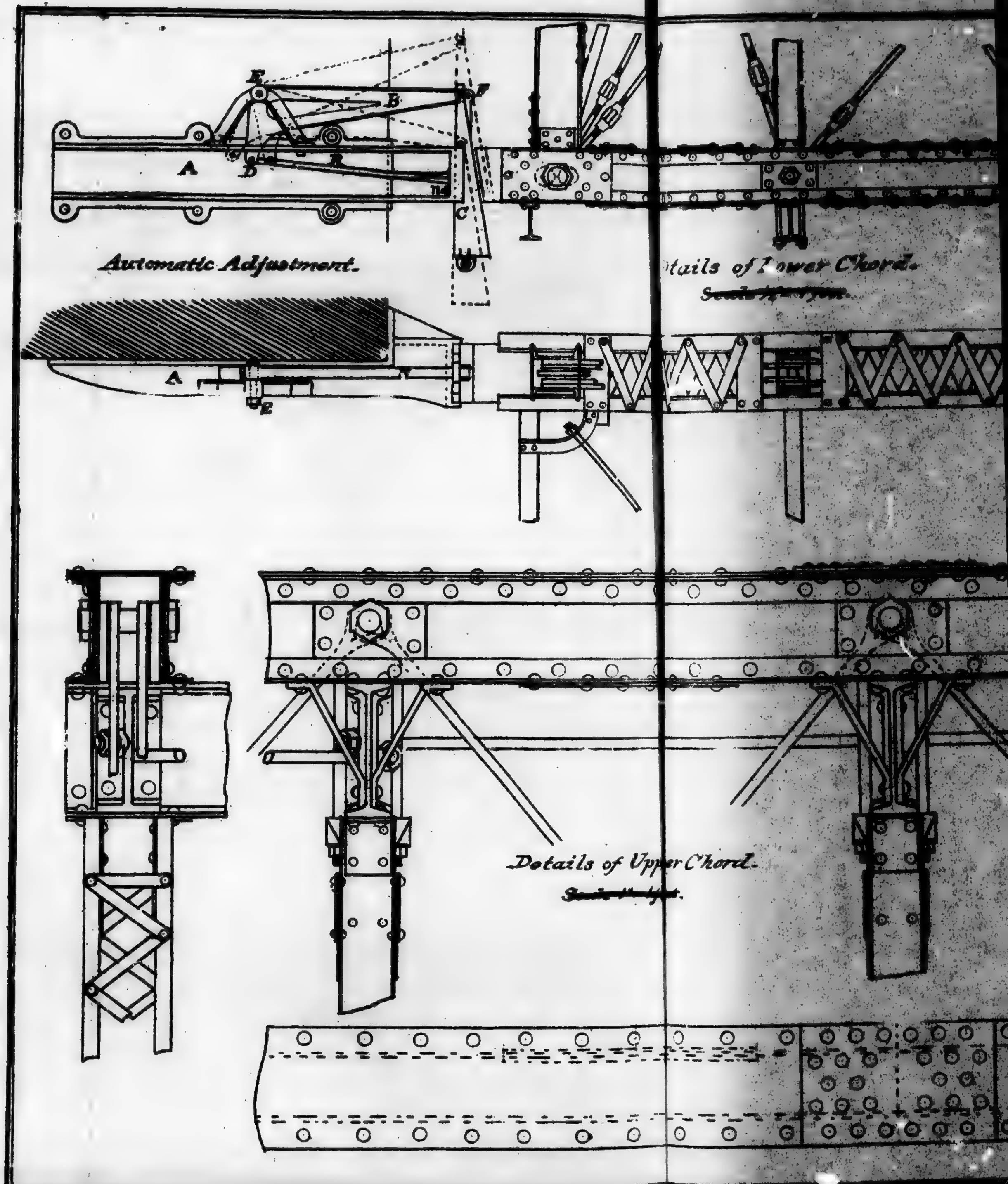
New York End.

NIAGARA
SUSPENSION BRIDGE.

[illegible].






Automatic Adjustment.
tails of Lower Chord.
Details of Upper Chord.

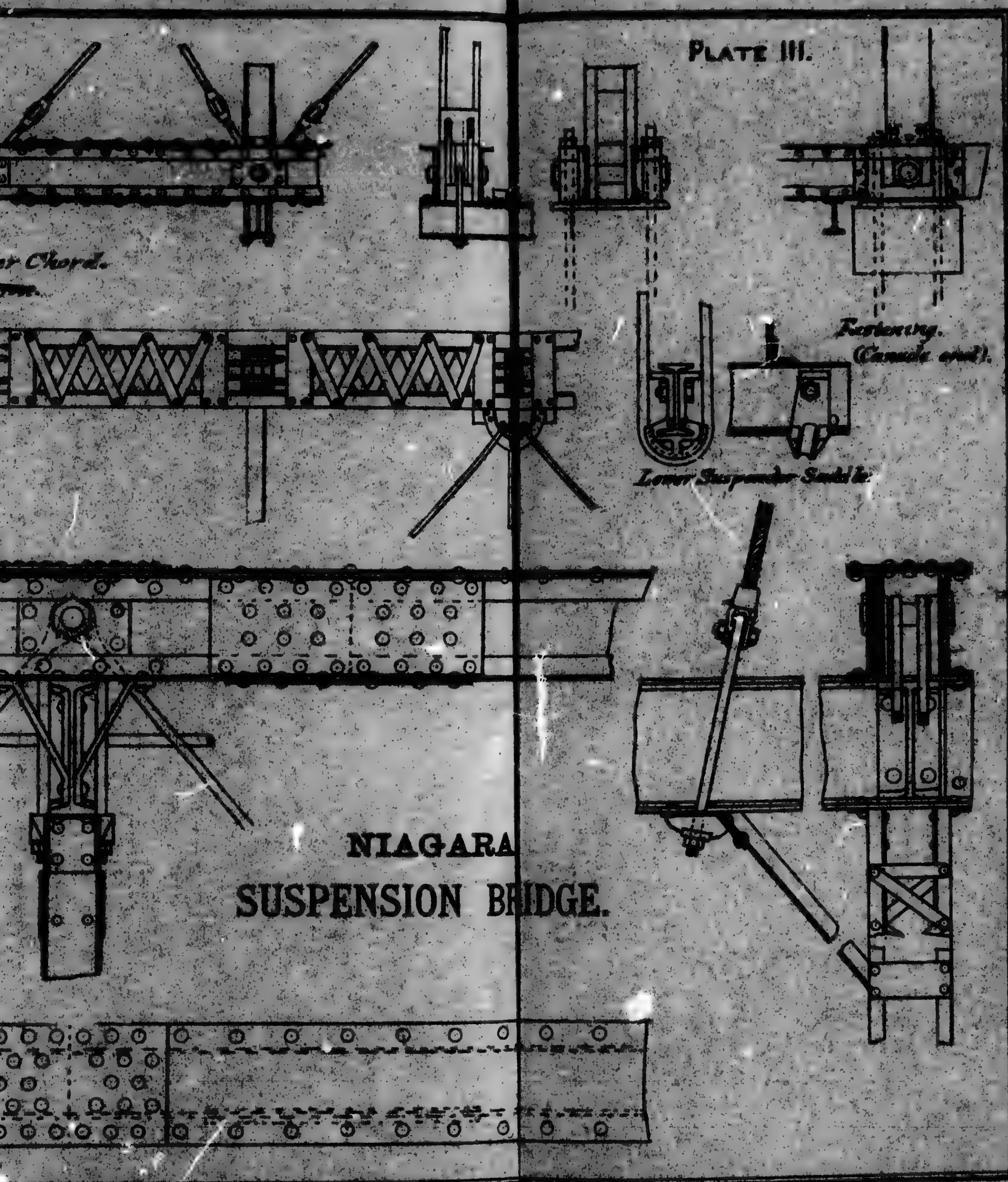
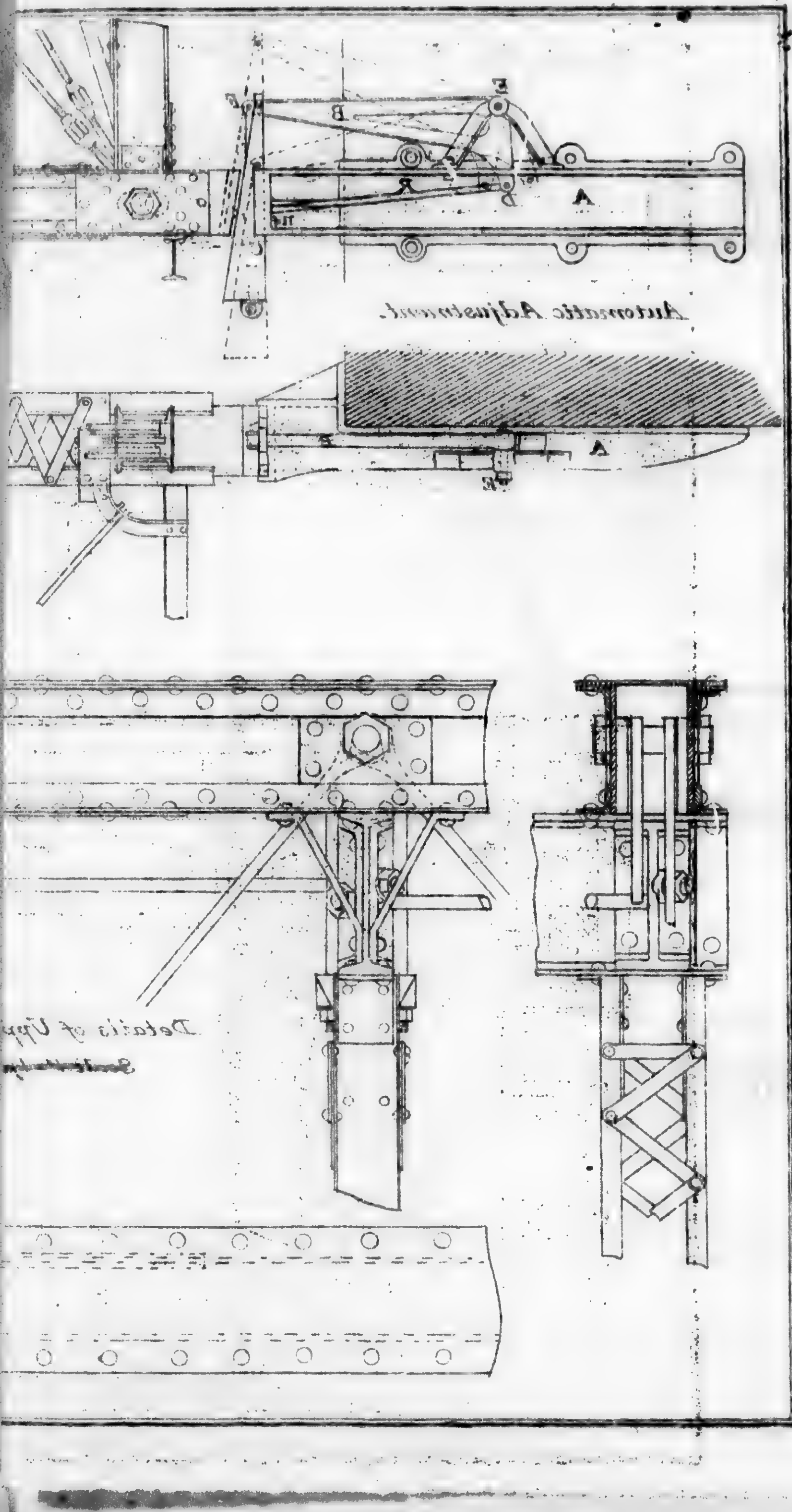
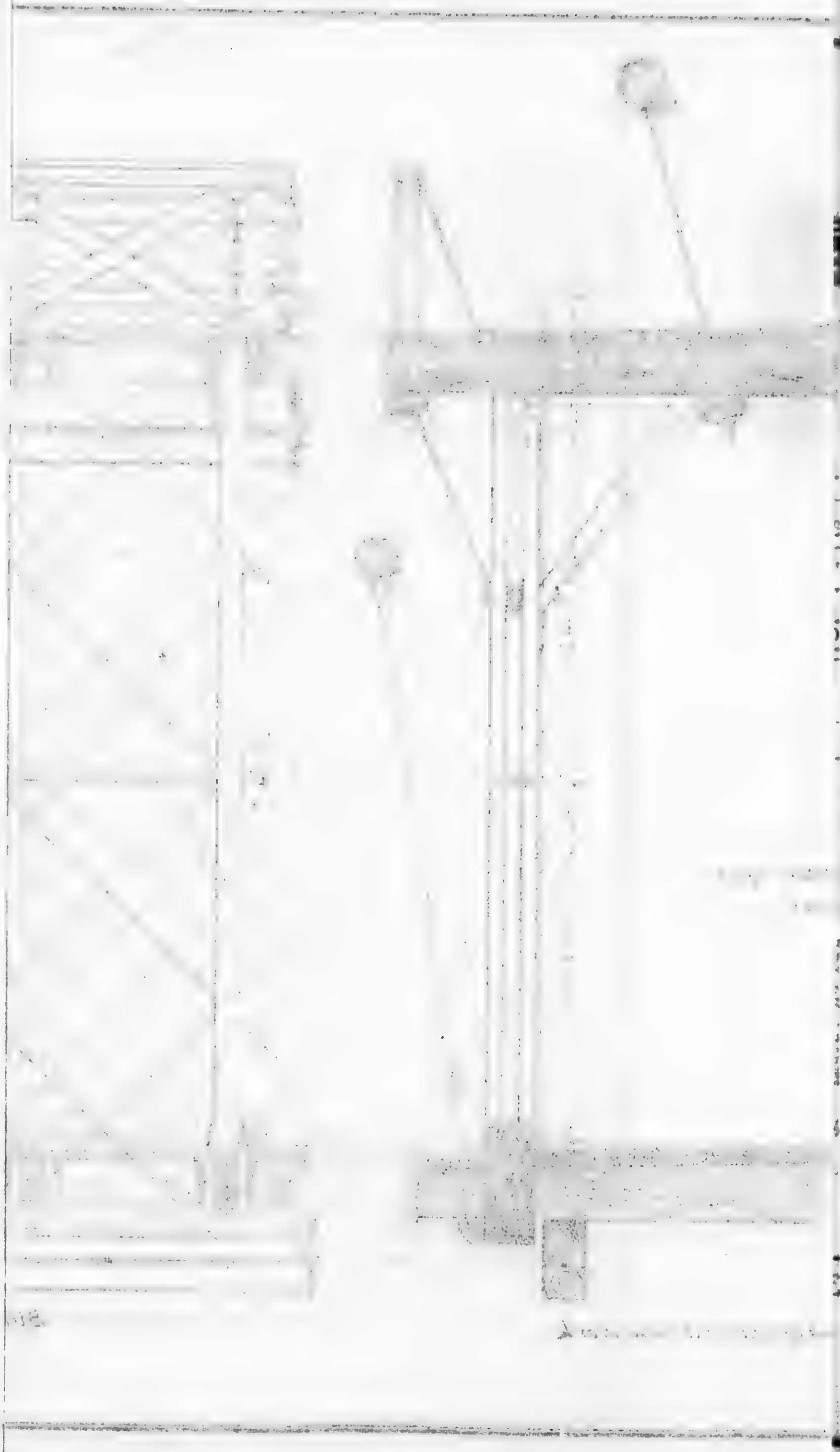
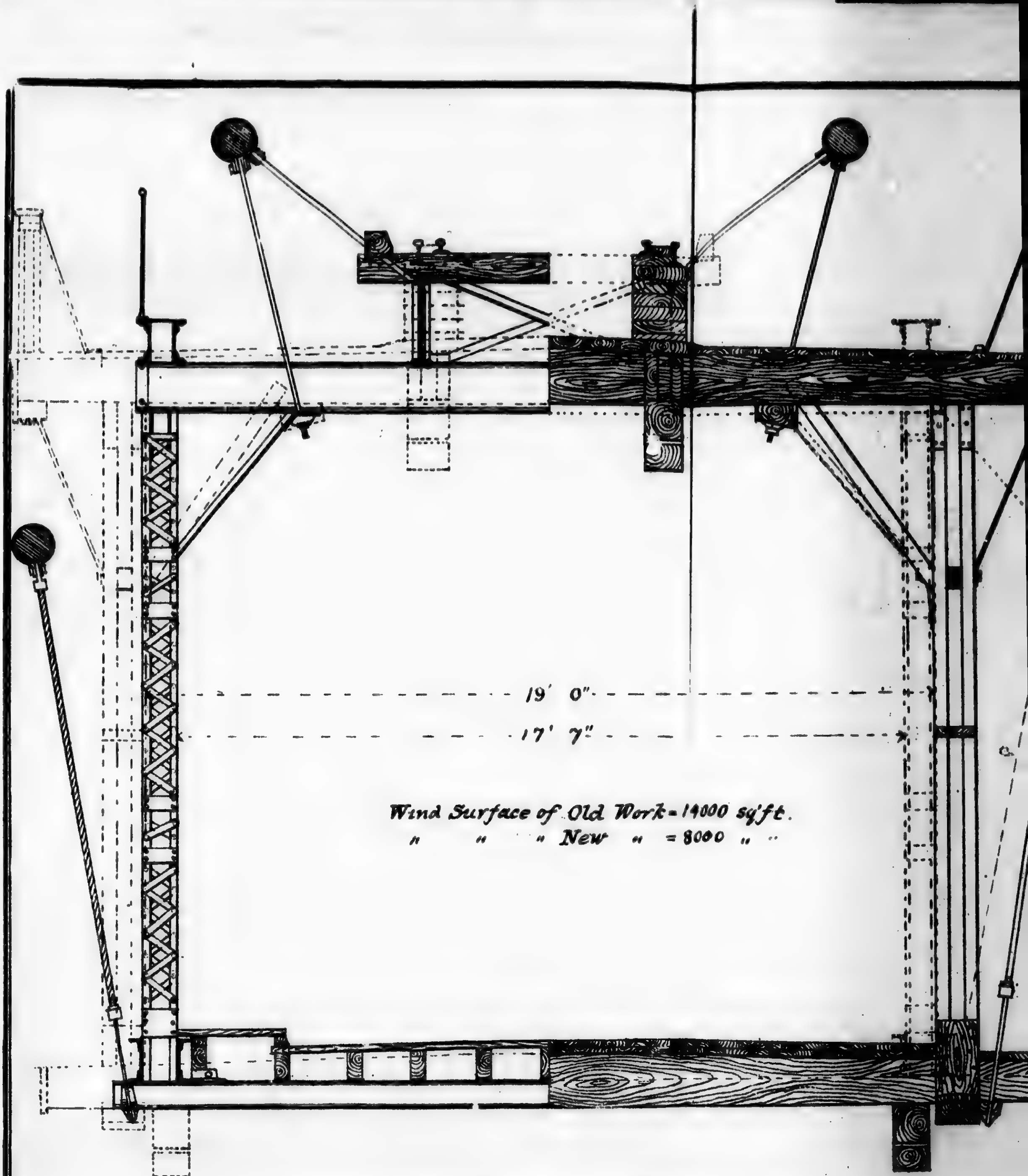


PLATE III.
Fastening.
(Canada end).
NIAGARA
SUSPENSION BRIDGE.

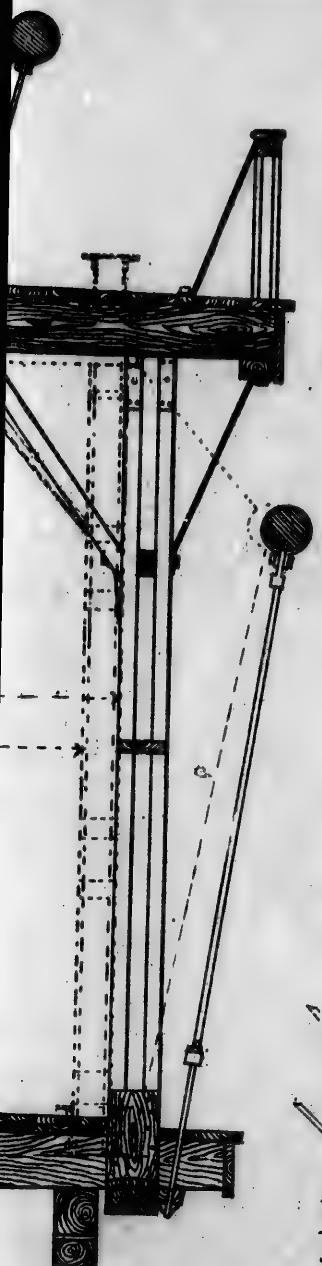

Automatic Adjustment.
Details of Upp



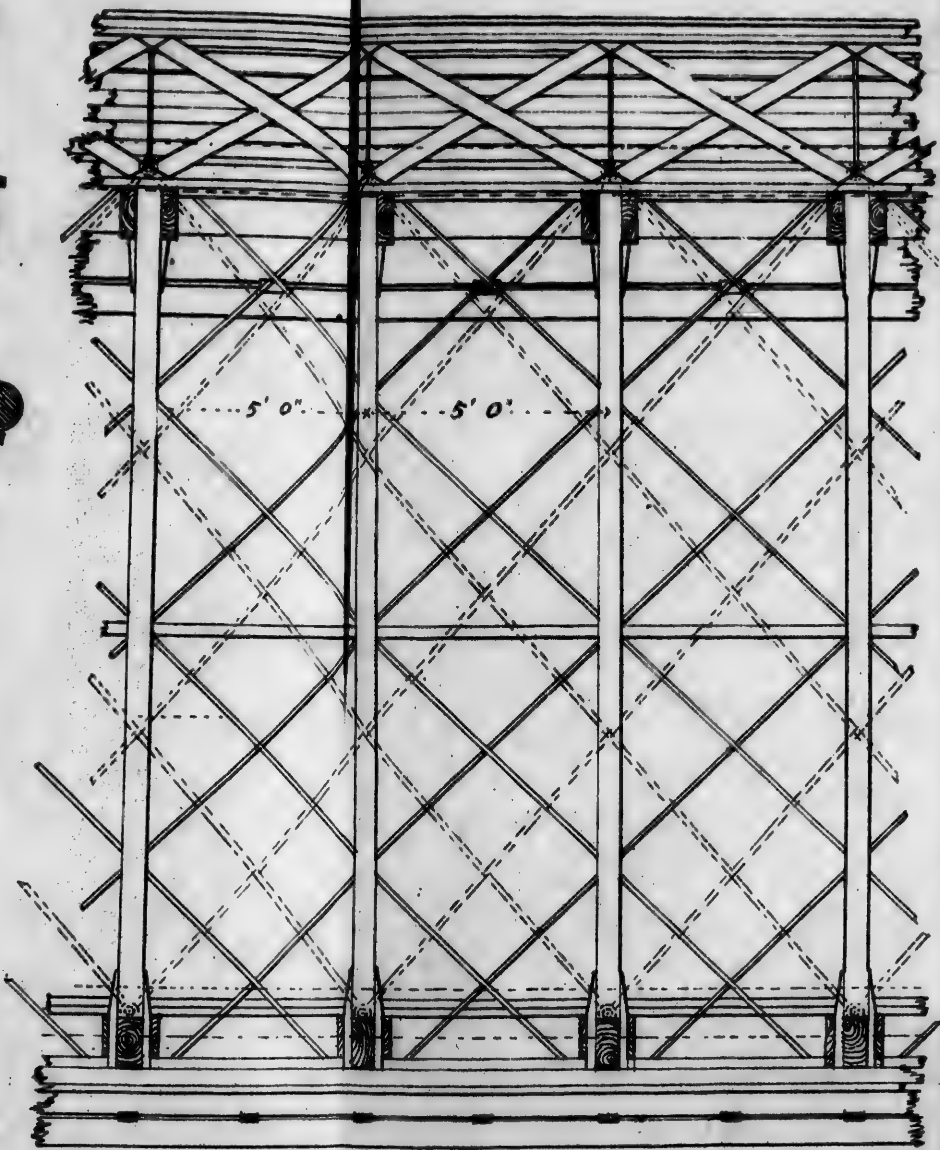


Transverse View showing relative positions of old and new work.

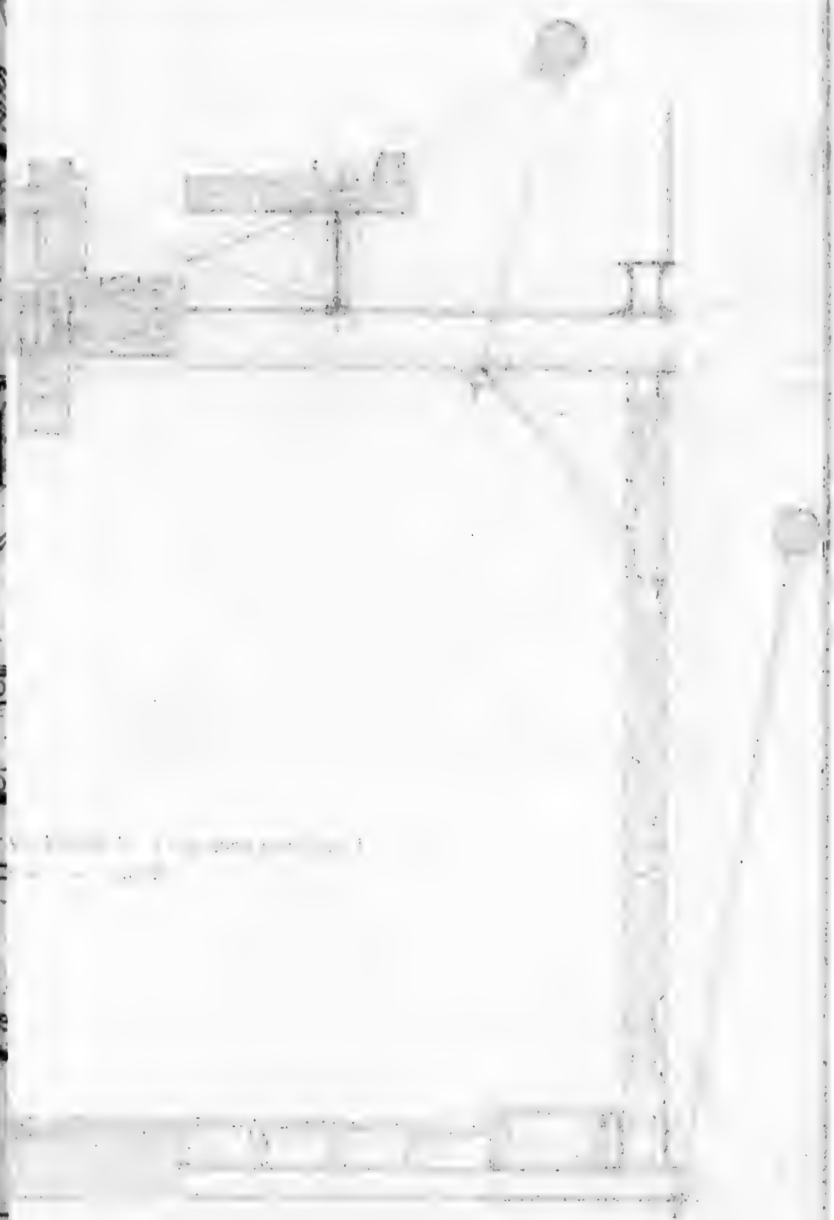
PLATE IV.



...w work.



Side Elevation of Old Truss.



300, 400 200

11000	10000	+2600	+3800	+3800	+4800	+3600

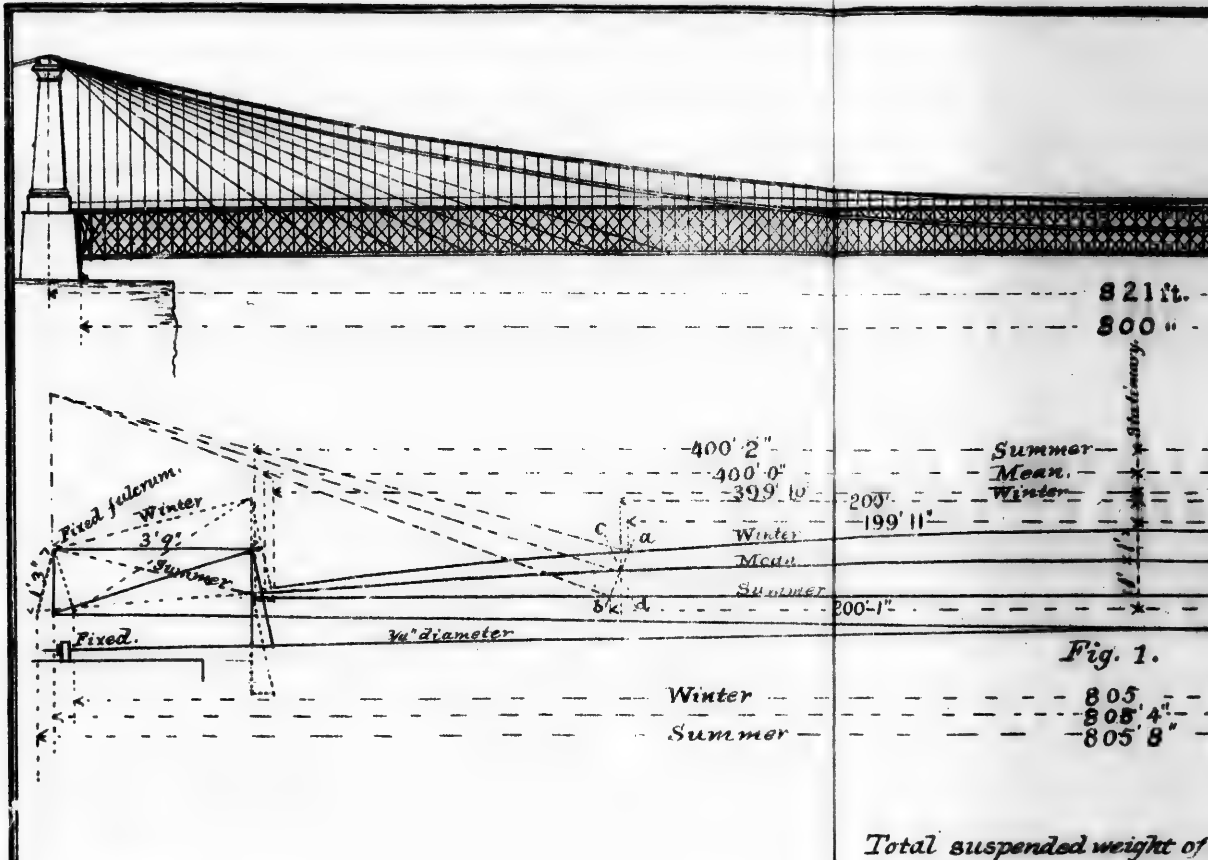
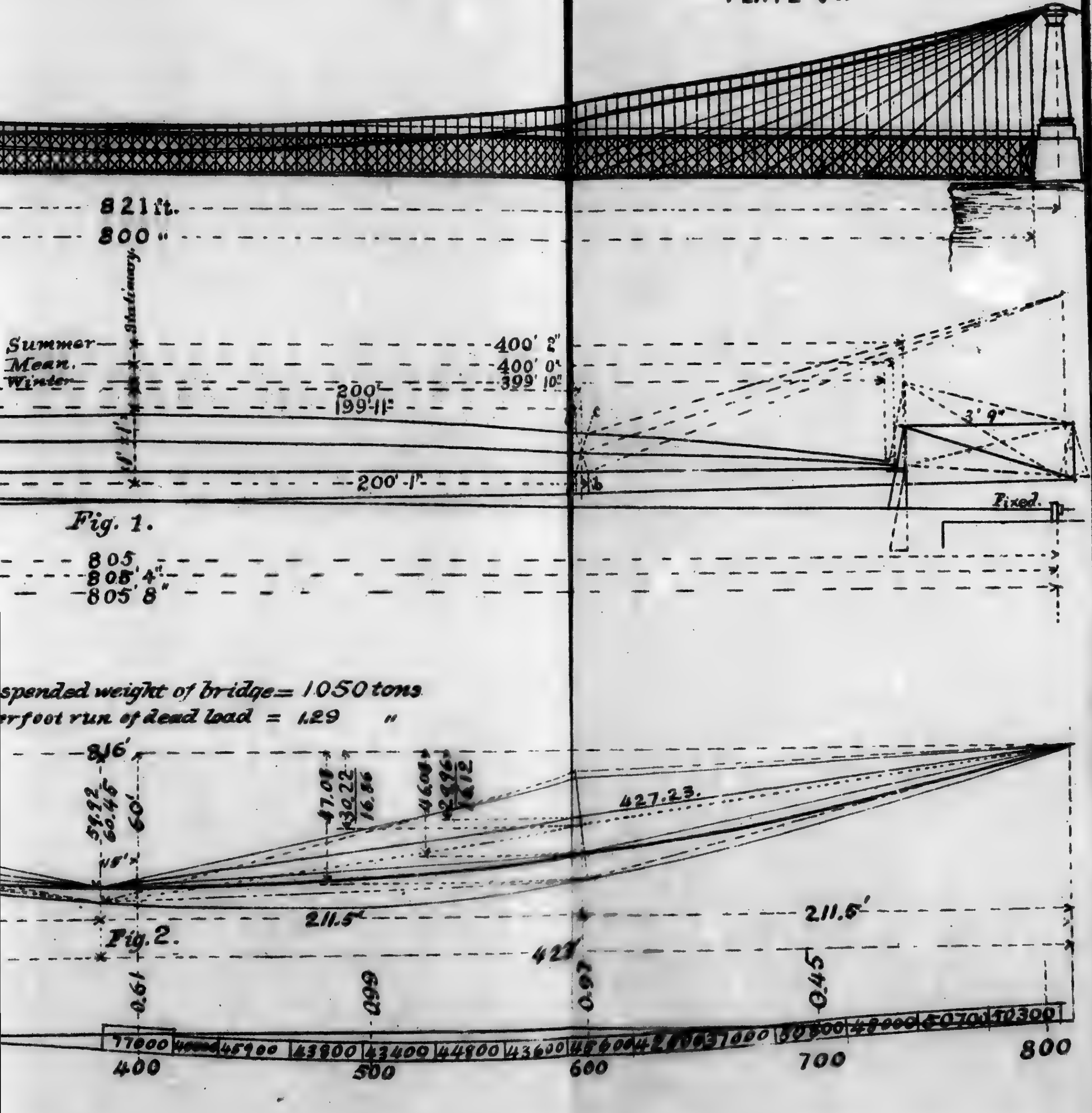
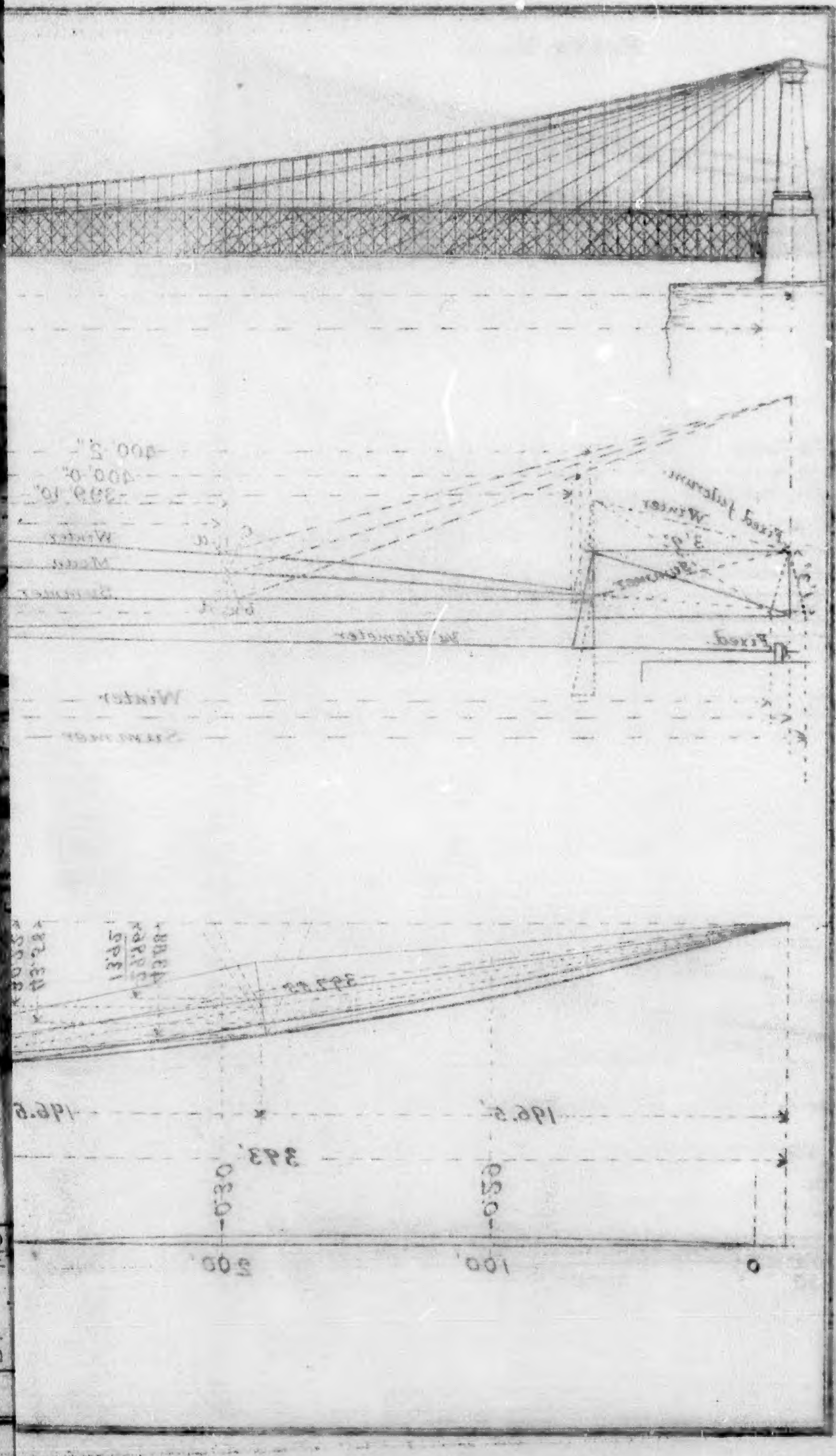

821 ft.
800 "
Stationary.
400' 2"
400' 0"
399' 10"
200'
199' 11"
Summer
Mean.
Winter
Fixed fulcrum.
Winter
3' 9"
Summer
1' 3"
c
a
b
k
d
Winter
Mean
Summer
200'-1"
Fixed.
3/4" diameter
Fig. 1.
Winter
Summer
805
805' 4"
805' 8"
Total suspended weight of
Weight per foot run of dead
816'
397.55
43.88
29.96
13.92
43.58
30.22
13.36
59.92
60.46
60'
196.5'
196.5'
393'
Fig. 2.
0.29
0.30
0.05
0.61
0
100'
200'
300'
400
77000

PLATE V.


821 ft.
800 "
Stationary
Summer
Mean.
Winter
400' 2"
400' 0"
399' 10"
200'
199'-11"
200'-1"
3' 9"
Fixed.
Fig. 1.
805
805' 4"
805' 8"
spended weight of bridge = 1050 tons
rfoot run of dead load = 1.29 "
816'
57.92
60.45
60'
47.08
30.22
16.86
46.08
28.96
14.12
427.23.
211.5'
211.5'
Fig. 2.
427
0.61
.099
0.97
0.45
77000
45900
43800
43400
44800
43600
37000
48000
50700
50300
400
500
600
700
800



400' 2"
400' 0"
399' 10"
Winter
Summer
Fixed fulcrum
Fixed
196.5'
393'
200'
100'
0

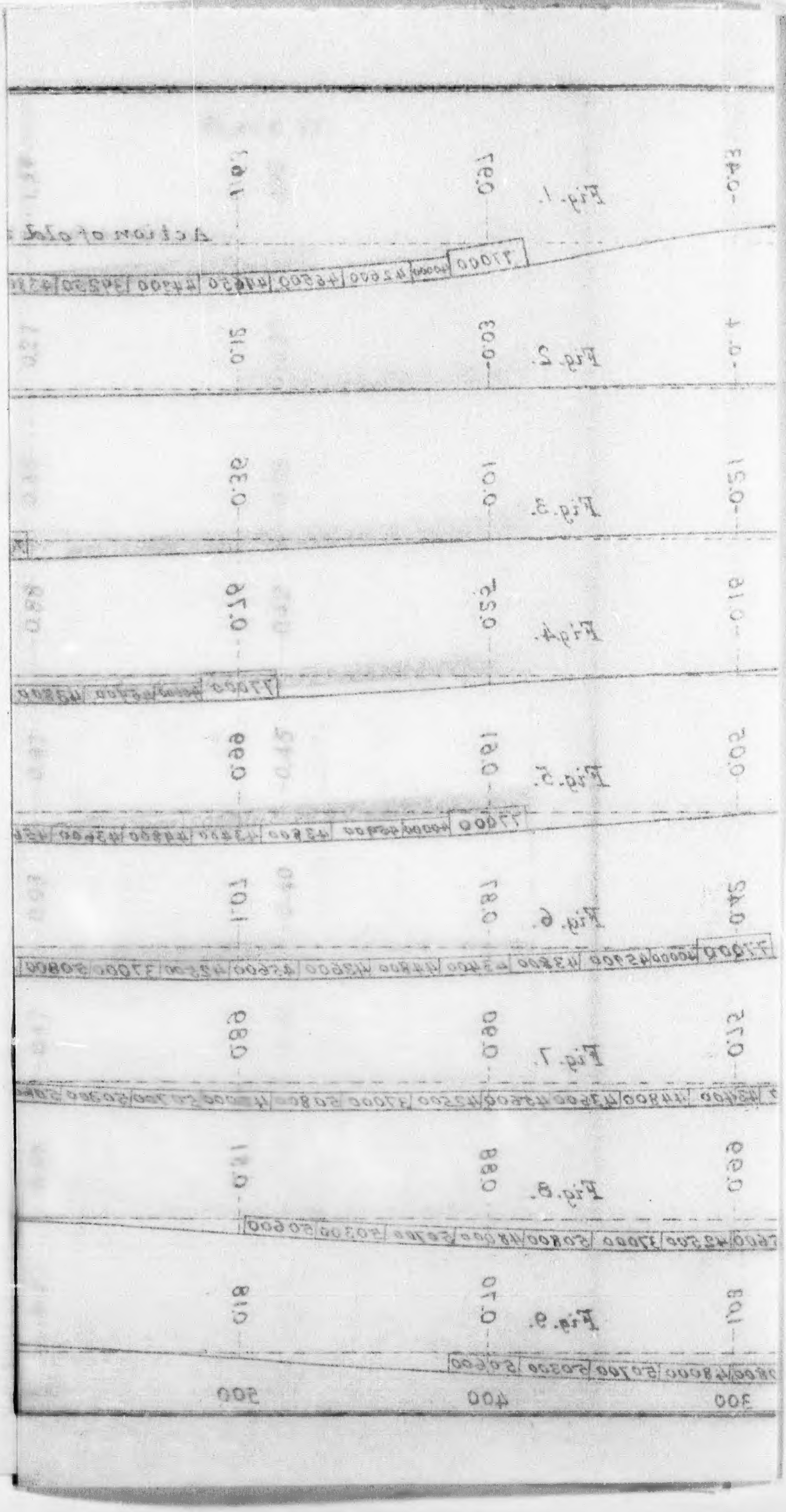


Fig. 1.
Fig. 2.
Fig. 3.
Fig. 4.
Fig. 5.
Fig. 6.
Fig. 7.
Fig. 8.
Fig. 9.
300
400
500

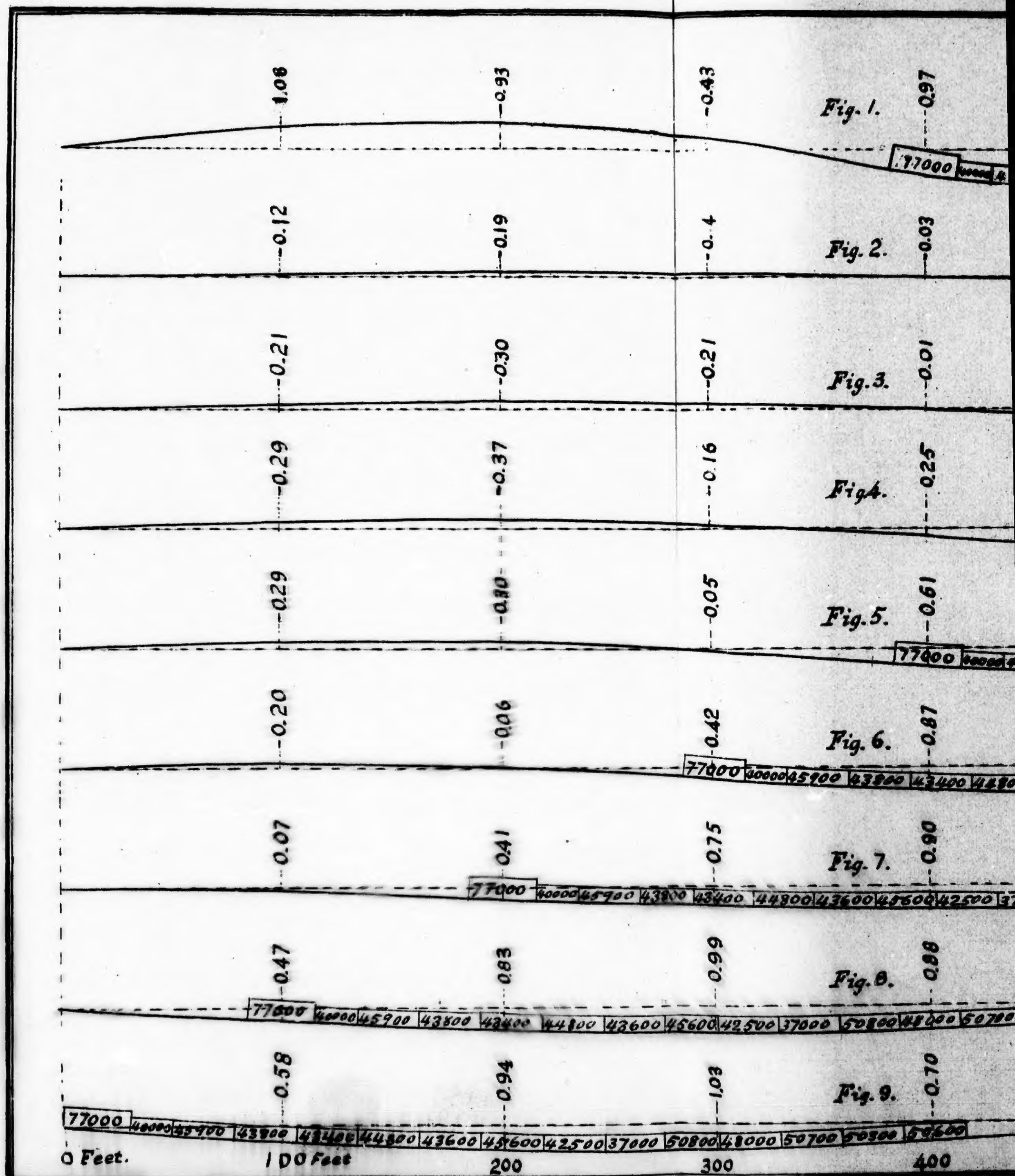


Fig. 1.
1.08
0.93
0.43
0.97
77000
Fig. 2.
0.12
0.19
0.4
0.03
Fig. 3.
0.21
0.30
0.21
0.01
Fig. 4.
0.29
0.37
0.16
0.25
Fig. 5.
0.29
0.30
0.05
0.61
77000
Fig. 6.
0.20
0.06
0.42
0.87
77000 40000 45900 43800 43400
Fig. 7.
0.07
0.41
0.75
0.90
77000 40000 45900 43800 43400 44800 43600 45600 42500
Fig. 8.
0.47
0.83
0.99
0.88
77000 40000 45900 43800 43400 44800 43600 45600 42500 37000 50800 48000 50700
Fig. 9.
0.58
0.94
1.03
0.70
77000 40000 45900 43800 43400 44800 43600 45600 42500 37000 50800 48000 50700 50300 50600
0 Feet.
100 Feet
200
300
400

PLATE VI.

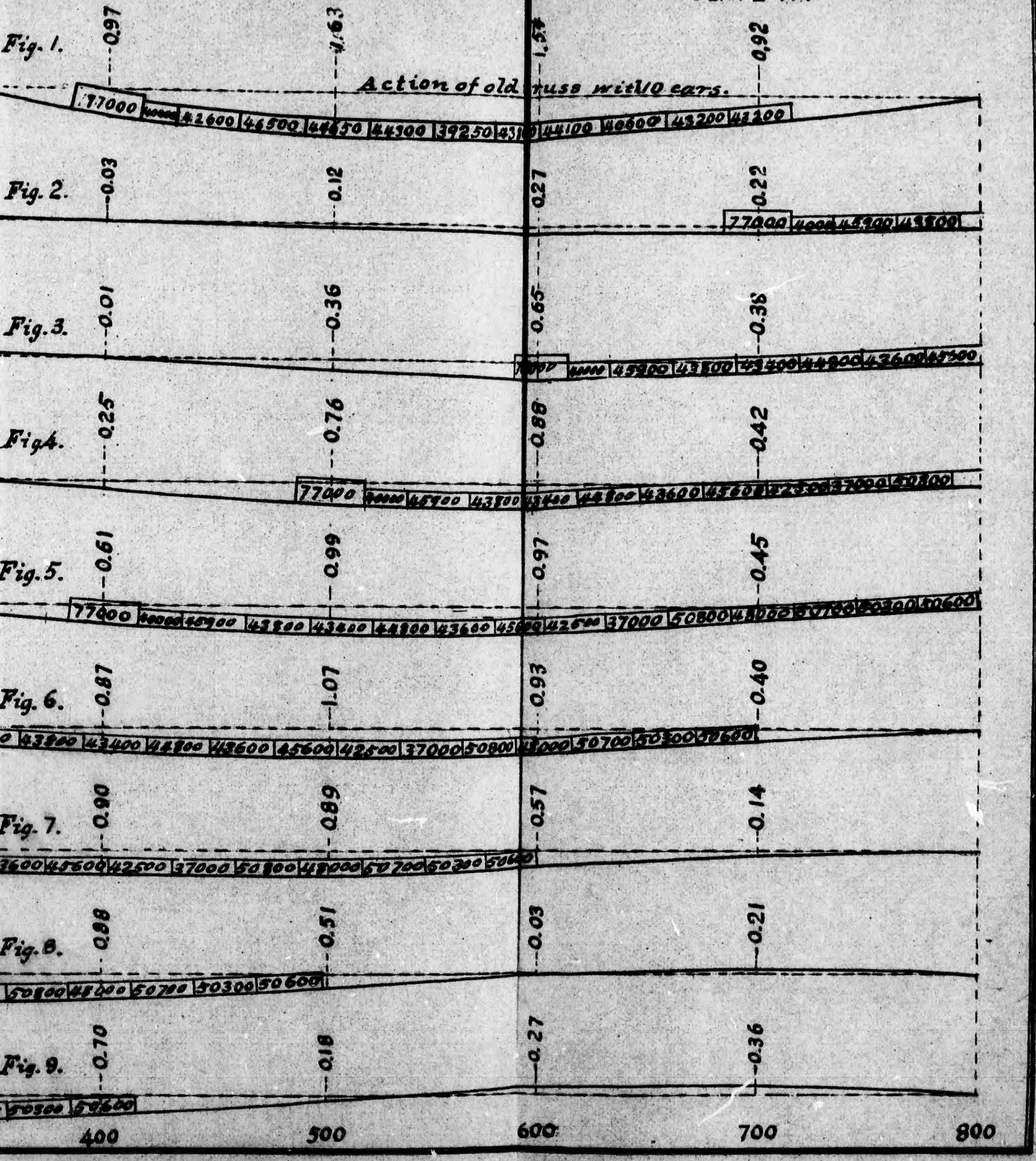


Fig. 1.
0.97
1.63
1.54
0.92
Action of old truss with 10 cars.
77000
42600
46500
44650
44300
39250
44100
40600
43200
Fig. 2.
0.03
0.12
0.27
0.22
77000
Fig. 3.
0.01
0.36
0.65
0.38
45900
43800
43400
44800
43600
Fig4.
0.25
0.76
0.88
0.42
77000
45900
43800
43400
44800
43600
45600
50800
Fig. 5.
0.61
0.99
0.97
0.45
77000
45900
43800
43400
44800
43600
45600
42500
37000
50800
48000
50700
50300
50600
Fig. 6.
0.87
1.07
0.93
0.40
43800
43400
44800
43600
45600
42500
37000
50800
48000
50700
50300
50600
Fig. 7.
0.90
0.89
0.57
0.14
45600
42500
37000
50800
48000
50700
50300
50600
Fig. 8.
0.88
0.51
0.03
0.21
50800
48000
50700
50300
50600
Fig. 9.
0.70
0.18
0.27
0.36
50300
50600
400
500
600
700
800